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ENGINEERING AND DESIGN

Coastal Groins and Nearshore Breakwaters

ENGINEER MANUAL

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	Engineering and Design COASTAL GROINS AND NEARSHORE BREAKWATERS	
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Engineer Manual
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COASTAL GROINS AND NEARSHORE BREAKWATERS

1. Purpose. This manual provides guidance for the design and placement of beach stabilization structures, specifically groins, nearshore breakwaters, and submerged sills.

2. Applicability. This manual applies to major subordinate commands, districts, laboratories, and field operating activities (FOA) having responsibility for the design of civil works projects.

3. General. Design of beach stabilization structures is complex. It requires analyses of the wave, current, and longshore transport environments and the coastal processes at a project site. It requires knowledge of the functional performance of the various shore stabilization schemes, the application of engineering judgment and experience to the design, and the structural design of a system that will withstand the marine environment and function as intended. Beach stabilization structure designs are site specific, and no single scheme is best for all situations; consequently, each design must be tailored to its specific objectives and site. This manual provides guidelines and design concepts but does not, in most cases, provide detailed design procedures.

FOR THE COMMANDER:

Milton Hunter
MILTON HUNTER
Colonel, Corps of Engineers
Chief of Staff

CECW-EH-D	**DEPARTMENT OF THE ARMY** **U.S. Army Corps of Engineers** **Washington, D.C. 20314-1000**	EM 1110-2-1617
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Table of Contents

Chapter 1 Introduction

1-1. Purpose and Scope

This manual provides guidance for the design and placement of beach stabilization structures, specifically groins, nearshore breakwaters, and submerged sills.

1-2. Applicability

This manual applies to major subordinate commands, districts, laboratories, and field operating activities (FOA) having responsibility for the design of civil works projects.

1-3. References

Required and related publications are listed in Appendix A.

1-4. Background

In highly developed beach communities, the consequences of previously ignored or unanticipated beach erosion may become costly enough to warrant using structural measures. Such measures may consist of seawalls, revetments, groins, bulkheads, breakwaters, and/or beach fills. Generally the "hard" structures require special siting considerations and an accompanying beach fill to mitigate adverse effects on adjacent beaches. Beach fills are often the preferred and sometimes the most cost-effective alternative. These "soft" structures include artificial beach berms and dunes accompanied by periodic beach nourishments, feeder beaches, or sand bypassing systems. Periodic or continuous replenishment of beach fills allows them to erode and adjust to the dynamic requirements of the ocean shore and prevent return of the damaging erosion processes to or beneath the landward development. Beach fills emulate nature, are aesthetically pleasing, contribute to recreation, and add needed beach material to the shore processes rather than simply redistributing available sand. An Engineer Manual on beach-fill design is in preparation at the US Army Engineer Waterways Experiment Station.

1-5. Discussion

a. Beach fills. Because beach fills are vulnerable to severe storms, they may be short-lived when a storm is experienced soon after the fill has been placed. This short existence is often viewed by the public as failure of the beach fill, even if the loss proves to be temporary. Little, if any, notice is given to the protection the fill provided to upland areas and the economic loss it may have prevented. Also, the sand may not necessarily have been lost, but may have been moved to an offshore bar. In some cases, the rising cost of sand placement is causing the economic viability of beach fills to decrease. In other cases, repeated beach fills have developed a public perception that beach fills and required periodic renourishments are wasteful. It is therefore politically and economically necessary to lengthen the interval between renourishments or rehabilitative beach fills, i.e., to increase the amount of time that placed sand remains on the beach. This increased longevity can be accomplished by the prudent design and placement of several types of beach stabilization structures. The design and placement of these structures, particularly groins, nearshore breakwaters, and submerged sills, is the subject of this Engineer Manual.

b. Protective and beach stabilization structures. A distinction is made between protective and beach stabilization structures. The purpose of the former is to protect inland development and to armor the shoreline against erosion; the purpose of the latter is to retard beach erosion, increase the longevity of a beach fill, and maintain a wide beach for damage reduction and recreation. Seawalls and revetments are shore protection structures whereas groins, nearshore breakwaters, and sills are beach stabilization structures.

1-6. Overview of Manual

The design of successful beach stabilization structures involves applying knowledge of the physical environment and coastal processes at a site to the selection of a type of structure, the preliminary design of that structure or structures, and the subsequent analysis and refinement of that design. The economic justification for beach stabilization structures is the savings realized by increasing the amount of time that nourishment sand remains on the beach within a project area. The cost of hard beach stabilization structures should be less than the beach nourishment savings realized. If, for example, including beach stabilization structures in a project increases the renourishment period from 3 to 6 years, the amortized savings accruing from the less frequent nourishment is available to build the structures.

a. Scope. Design of beach stabilization structures is complex. It requires analyses of the wave, current, and longshore transport environments and the coastal processes at a project site. It requires knowledge of the

functional performance of the various shore stabilization schemes, the application of engineering judgment and experience to the design, and the structural design of a system that will withstand the marine environment and function as intended. Beach stabilization structure designs are site specific, and no single scheme is best for all situations; consequently, each design must be tailored to its specific objectives and site. This manual provides guidelines and design concepts but does not, in most cases, provide detailed design procedures. References to the source of detailed design procedures are cited where appropriate.

b. Chapter 2. Chapter 2 provides general design considerations for beach stabilization structures, alternative types of beach stabilization structures, the various types of construction, and the general data requirements for design including wave and water-level data, longshore sand transport data, and shoreline change data.

c. Chapter 3. Chapter 3 deals with the functional and structural design of groins and groin systems. Groin dimensions such as height, length, spacing, and permeability, and their effects on a groin's functional performance are discussed along with the use of physical and mathematical models to evaluate designs. Wave, current, and earth forces on groins are also discussed.

d. Chapter 4. Chapter 4 deals with nearshore breakwaters, artificial headlands, and submerged sills. Design objectives are outlined along with descriptions of single and multiple nearshore breakwaters, artificial headlands, and submerged sills. Design factors include selecting the desired shoreline configuration and the breakwater height, length, distance from shore, permeability, spacing, and type of construction that will achieve the desired effect. The effect of breakwaters on nearshore circulation, wave conditions in the breakwater's lee, longshore transport, and onshore-offshore transport are discussed.

e. Chapter 5. Chapter 5 deals with construction and postconstruction activities, specifically, construction records, inspections, and project monitoring. Monitoring data include: ground photography, aerial photography, inspection reports, beach and dune profile surveys, wave data, other environmental data, wave force data, and ecological and archeological data. Requirements of the Operations and Maintenance Manual that must be developed to assist local sponsors in properly operating beach stabilization projects are discussed. This manual is required under ER 1110-02-1407.

f. Appendixes. Appendix A is a list of references cited. Appendix B is a compilation of the advantages and disadvantages of the various types of beach stabilization systems. Groins, nearshore breakwaters, submerged sills, and alternative beach stabilization schemes are considered. Appendix C describes dimensional analysis related to groin design and provides an example application. Appendix D provides a description of the numerical shoreline change model GENESIS. Appendix E provides a dimensional analysis for breakwater and submerged sill design and provides an example application for a detached breakwater.

Chapter 2
Design Considerations for Beach Stabilization Structures

2-1. General Design Objectives

a. Structural versus nonstructural alternatives.

(1) Beach stabilization structures alone do not provide the sand to maintain a wide protective or recreational beach; they simply redistribute available sand. Thus, accretion in one area is balanced by erosion elsewhere unless additional sand is introduced into the project area. The design of shore protection without concomitant beach nourishment must recognize that more sand in one area often means less in another area. The degree of allowable adverse effects needs to be addressed; however, if negative impacts cannot be tolerated, beach nourishment must be included in the project.

(2) Beach and dune restorations are often vulnerable and short lived due to the frequency and intensity of coastal storms. In addition to providing protection, however, they also contribute additional sand to the littoral environment. Frequent renourishment may be necessary to maintain a given level of protection. Coastal structures placed in conjunction with beach nourishment can often increase the residence time of the sand, keeping it on the beach within the project area for a longer period of time. If the savings realized by reducing the time between required renourishment exceeds the cost of the structures, their construction can be justified.

b. Alternative types of beach stabilization structures.

(1) Shore-parallel, onshore structures. Several types of beach stabilization structures can be built parallel to shore on an existing or restored shoreline. Revetments, bulkheads, and seawalls protect the area immediately behind them, but afford no protection to adjacent areas nor to the beach in front of them. While revetments, bulkheads, and seawalls can modify coastal processes such as longshore transport rates, cross-shore distribution of longshore transport, and onshore-offshore transport on the beach in front of them (if they protrude into the zone of longshore transport), these modifications do not affect their intended function, which is to protect the property behind them. These structures stabilize a shoreline by enclosing and protecting an area, thereby preventing the beach from functioning normally. The function and design of revetments, bulkheads, and seawalls is discussed in EM 1110-2-1614.

(2) Shore-connected structures.

(a) Groins and shore-connected breakwaters comprise the two types of beach stabilization structures in this category. Groins are the most common shore-connected beach stabilization structures. They are usually built perpendicular to shore to interrupt the normal transport of sand alongshore. Wave-induced longshore currents move sediment and cause it to accumulate in a fillet along the groin's updrift side (the side from which the sediment is coming). The groin also shelters a short reach of shoreline along its downdrift side from wave action. The accumulation of sand in a fillet along the updrift side of the groin reorients the shoreline and reduces the angle between the shoreline and the prevailing incident waves. This reduces the local rate of longshore sand transport and results in accumulation and/or redistribution of sand updrift of the groin and a reduction in the amount of sand moving past the groin. Diminished sand transport past a groin reduces the amount of sand contributed to the downdrift area and often causes erosion. Frequently, several groins are spaced along a beach to stabilize a long reach of shoreline. The groin system may or may not include a beach fill. If not artificially filled, natural longshore transport processes must fill the system. During the time the groins are filling, sand transport to downdrift beaches will be significantly reduced. This interruption of the natural sediment supply will cause erosion at the downdrift beaches. Unless special conditions warrant, prefilling the groin system should be considered mandatory.

(b) While groins are most often shore-perpendicular, they may sometimes be hooked or curved, or they may have a shore-parallel T-head at their seaward end. Hooked or curved groins are built in an attempt to increase the size of the updrift fillet or to shelter a greater stretch of beach from storm waves approaching from a predominant direction. A T-head groin may function primarily as a groin or as an offshore breakwater depending on the length of the T-head, structural transmissibility, and distance from shore. The T-head is often built to interrupt the seaward flow of water and sand in rip currents that often develop along a groin's axis. The T-head may also act as a breakwater and shelter a sizeable stretch of beach behind it.

(c) Important parameters that must be determined in designing a groin or groin system include: length, height and profile, planform geometry, spacing alongshore, type and materials of construction, permeability to sand, and the proposed fill sand's gradation.

(d) Shore-connected breakwaters extend seaward from shore and protect a stretch of beach from wave action. The quiet water behind the breakwater precludes erosion and, if sediment is in transport, allows it to accumulate in the structure's lee. Shore-connected breakwaters are generally dog-leg shaped in plan with a shore-connecting leg and a nearly shore-parallel leg; the shore-connecting leg often functions like a groin. They are often of either rubble-mound or sheet-pile construction. Frequently, shore-connected breakwaters are built to provide shelter for a marina rather than to provide shore stabilization. Shore stabilization and sedimentation effects are secondary, and the resulting sedimentation is often unwanted.

(3) Nearshore, shore-parallel breakwaters.

(a) Shore-parallel, detached (not shore-connected) breakwaters may be built singly or in series spaced along the shoreline. Detached breakwaters are constructed close to shore to protect a stretch of shoreline from low to moderate wave action and to reduce severe wave action and beach erosion. Sand transported along the beach is carried into the sheltered area behind the breakwater where it is deposited in the lower wave energy region. Protection afforded by the breakwater will limit erosion of the salient during significant storms and promote growth during periods of low to moderate wave activity. The effectiveness of a nearshore breakwater or breakwater system depends on the level of wave protection and the length of the shoreline it protects; thus, the breakwater's height, length, wave transmission characteristics, and distance from shore contribute to its effectiveness. For a system of breakwaters, the width of the gap between adjacent breakwaters and the length of the individual breakwater segments are also important.

(b) Nearshore breakwaters can also be constructed to create artificial headlands and are referred to as artificial headland breakwaters. In nature, where headlands are closely spaced and a limited sediment supply exists, small pocket beaches are formed (Chew et al. 1974). Pocket beaches are in hydraulic equilibrium, inherently stable, and recover rapidly after storm events (Hardaway and Gunn 1991). Where natural headlands are far apart and an adequate sediment supply exists, long and wide beaches are formed. Most headland beaches are between these extremes and assume a shape related to the predominant wave approach: a curved upcoast section representing a logarithmic spiral and a long and straight downcoast section (Chew et al. 1974). Headland beaches are often termed log-spiral beaches, crenulate-shaped, or pocket beaches. As opposed to detached breakwaters where tombolo formation is often discouraged, an artificial headland breakwater is designed to form a tombolo. Artificial headland design parameters include the approach direction of dominant wave energy, length of individual headlands, spacing and location, crest elevation and width of the headlands, and artificial nourishment.

(4) Shore-parallel offshore sills (perched beaches).

(a) Submerged or semisubmerged, shore-parallel offshore sills have been suggested as shore protection structures that can reduce the rate of offshore sand movement from a stretch of beach. The sill introduces a discontinuity into the beach profile so that the beach behind it is at a higher elevation (and thus wider) than adjacent beaches. The beach is thus "perched" above the surrounding beaches. This sill acts as a barrier to reduce offshore sand movement and causes some incoming waves to break at the sill. The sill functions like a nearshore breakwater by providing some wave protection to the beach behind it, although this sheltering effect is generally small since the sill's crest is relatively low. The height of the sill's crest and its alongshore continuity differentiates submerged sills from nearshore breakwaters. The crest of the submerged sill is usually continuous and well below normal high-tide levels; in fact, it is usually below low-tide levels.

(b) The low sill/perched beach concept minimizes the visibility of the structure since the sill crest is below the water's surface most of the time. Even when visible at low tide, it often remains more aesthetically acceptable than a detached breakwater. A disadvantage of the sill, however, is its potential as a hazard to swimming and navigation.

(5) Other. In many coastal locations, shore stabilization structures are already in place, having been built in response to a continuing erosion problem. These structures often have been modified over the course of their lifetime in attempts to improve their performance or to mitigate any adverse effects they might have caused. These modifications often account for the strange configurations of many structural shore stabilization systems found along eroding shorelines. For example, groins may initially have been built and subsequently modified by the addition of spurs (a diagonal extension off the structure),

hooked sections, or T-heads to reduce offshore sediment losses. Multiple-groin systems may have been extended downdrift along the coast in response to the progressive downdrift displacement of an erosion problem due to reducing the natural sand supply by updrift groin construction.

c. *Selection among alternatives.* Three major considerations for selecting among alternative beach stabilization schemes are: the primary and secondary objectives of the project, the physical processes prevailing at the project site, and the potential for adverse impacts along adjacent beaches. Appendix B provides descriptions of some of the advantages and disadvantages for various beach stabilization schemes.

(1) Primary and secondary objectives. Several factors determine what measures best meet the objectives of a given project. An important first step in selecting among alternative stabilization schemes is to carefully define the project's primary objective and any secondary objectives.

(a) A project's primary objective may be to protect inland development, maintain a beach, or both. Structures that armor the shoreline, beach stabilization structures, beach nourishment, or a combination of these may satisfy a project's primary objective. If the objective is simply to protect inland development from storm damage and to armor the shoreline against further erosion, a purely hard structural solution using a revetment or seawall might suffice. A beach seaward of the protective structure may or may not be important. If the objective is to protect inland development while maintaining a beach for additional protection and/or recreation, a solution involving either shore protection structures fronted by a beach fill, beach fill alone, or beach fill with stabilization structures might be sought. If the primary objective is to provide a protective beach or to stabilize an existing beach, then beach fill alone or beach fill with stabilization structures may be the solution.

(b) Secondary project objectives should also be identified and can often lead to additional project benefits. For example, a project's primary objective may be protection; however, a wide protective beach may also provide recreational benefits. Similarly, a project's primary objective may be to maintain a recreational beach, which will also afford some protection to back-beach development.

(2) Physical processes. Selecting an alternative shore protection/beach stabilization scheme also depends on the physical processes that prevail at a project site. If beach stabilization is a project's primary objective and net sediment losses from the project area are mainly by longshore transport, groins may provide a solution. On the other hand, if sediment losses are primarily offshore, groins cannot slow offshore losses; but, may exacerbate offshore diversion of sand by inducing rip current formation. Nearshore breakwaters reduce both alongshore and offshore sand losses, but significantly reduce wave conditions along the beach. Lower surf may or may not be desirable depending on intended beach use.

(3) Adverse impacts along adjacent beaches. The effect of a project on adjacent beaches is also a factor in selecting from among various types of shore stabilization. Structures such as groins and nearshore breakwaters, which reduce or for a time totally halt longshore transport, can cause erosion both downdrift and updrift of a project area. This impact can be avoided or mitigated by including beach nourishment as a part of the project. Including beach fill reduces the time it takes for the project to establish a new equilibrium beach planform configuration. It can take several years for a new equilibrium to be established if sand must be supplied by natural longshore sand transport alone. Beach fill thus encourages earlier sand bypassing of the project and reduces downdrift erosion. Where possible, groins and nearshore breakwaters should be designed to allow some sand bypassing to help alleviate downdrift erosion. If downdrift erosion is of no concern (such as the downdrift end of an island or a beach adjacent to a rocky shore), groin compartments and the beach behind nearshore breakwaters can be allowed to fill by natural longshore transport, if sufficient sediment is naturally available.

(a) Groins. Groins control the rate of longshore sand transport through a project area and reduce the rate of sand lost alongshore to downdrift beaches. If properly designed, they are effective in stabilizing beaches where sand is lost by alongshore movement. Groins function regardless of the direction of longshore transport and may exhibit seasonal variations in the location of the sand fillet as it shifts from one side of the structure to the other depending on the prevailing wave direction. Their effects often occur some distance both updrift and downdrift of the structure. Thus a single, relatively small groin can accumulate sand along a relatively long stretch of shoreline; likewise, erosion effects can often occur some distance downdrift of the structure. Groins are relatively easy to construct using land-based construction equipment and are also relatively easy to inspect and maintain. Groins do not significantly alter the characteristics of the waves along the beach except for a relatively limited area near the groin itself. They may cause offshore losses of

sand during periods of high waves and water levels by deflecting longshore currents seaward. Wave setup in the compartment between two groins is greater on the updrift side of the downdrift groin, since waves there are larger and the shoreline is not sheltered by the structure. This condition causes a circulation within the compartment and may cause a rip current along the groin that can carry sand seaward. If sand losses from a beach are by offshore movement, groins will be ineffective in controlling erosion. Like all structures, groins alone do not provide sand; they simply redistribute available sand. Thus, sand held in an updrift fillet is kept from downdrift beaches, resulting in increased downdrift erosion rates. This problem can be avoided or delayed by including beach fill and nourishment as part of a groin project.

(b) Nearshore breakwaters. Nearshore breakwaters are effective shoreline stabilization structures that control both alongshore and offshore movement of sediment. They can be designed either singly or as a system of segmented breakwaters depending on the length of shoreline to be protected. There has been limited US experience with nearshore breakwater design, construction, and performance; thus, there is limited documented experience on which to base a design. The amount of longshore transport moving along a beach can be controlled by adjusting the length and spacing of the breakwater segments; however, unless the segments are carefully designed, nearshore breakwaters can disrupt longshore transport and starve downdrift beaches. Also, if built too close to shore, a tombolo (a sand spit extending from shore out to the offshore breakwater) can develop. The tombolo and breakwater can act as a groin, creating a total block to longshore sand transport until a new equilibrium is reached and bypassing resumes. Nearshore breakwaters significantly change the nature of the surf zone and the characteristics of the waves along a beach. Large waves break seaward of the breakwaters and only low, diffracted waves reach the beach behind the breakwaters. Waves acting on the structure may cause toe scour on the seaward side, and since the structures are located in shallow water nearshore, they are often subjected to the full force of breaking waves. Design wave conditions may be more severe than for revetments and seawalls onshore. Nearshore breakwaters are relatively expensive to construct because of their offshore location. Construction can be from the water using barges, from a temporary trestle, or from a temporary embankment built out from shore to the breakwater site. This embankment may later become part of a beach fill associated with the project. Likewise, inspection, maintenance, and repair will be more difficult and expensive than for land-connected structures.

(c) Beach fill. Beach fill and periodic nourishment are the only solutions to beach erosion problems that actually provide additional sand for a beach. Fill sand is usually obtained from a location some distance from the nourished beach: either an inlet, backbay area, or, in recent years, offshore or imported sources. It is often coupled with other shore protection measures to provide additional protection and recreation. Beach fills are often designed to provide a protective beach--a barrier of sand between the ocean and any back-beach development. Unless measures are taken to retain the beach fill and increase its residence time within a project area, beach fills may be short-lived. The presence of the fill does not appreciably alter the wave and nearshore current environment, and thus the erosion-causing factors continue unabated. Periodic nourishment is necessary to maintain a given level of protection. Depending on the size distribution of the fill sand relative to the native sand, erosion of the beach fill may be faster or slower than the original prefill erosion rate. Beach-stabilizing structures are built in conjunction with beach-fill projects to increase the residence time of the sand within the project area. As nearby sources of good quality beach sand are depleted, the cost of beach nourishment will increase since more distant sources must be exploited. Because of increasing costs of fill, stabilizing structures are becoming more economical. Structures are justified if they decrease the frequency of required periodic nourishment (increase the residence time between fills) so that nourishment is required less often. The anticipated savings accrued by less frequent nourishment should exceed the cost of structures.

d. Types of construction. Beach stabilization structures may be built of various materials and in various configurations. Factors such as the functional performance, cost, durability, and expected functional lifetime of an installation determine what type of construction is best.

(1) Rubble-mound construction. Groins, breakwaters, and offshore sills (perched beaches) are commonly constructed of quarrystone. Generally, rubble-mound structures comprise the most common type of coastal construction because they are able to dissipate most incident wave energy, thus reducing wave transmission and reflection. They are also "flexible" structures that do not lose their ability to function even when occasionally subjected to waves larger than the conditions for which they were designed. Failure is usually slow and progressive rather than catastrophic as it might be for more rigid structures.

(a) The design of rubble structures is described in EM 1110-2-2904 and the *Shore Protection Manual* (SPM 1984). Basically, the structure's outer or armor layer is built of quarrystone large enough to withstand selected design wave conditions at a selected design water level. The first underlayer (the layer of stone beneath the armor layer) is sized large enough so that it will not fit through the voids between the elements of the overlying layer. Each successive underlying layer is just large enough to be retained under the layer above it until quarry-run stone can be used in the cores. Armor stone is carefully placed and keyed to achieve maximum stability; however, it should be placed with sufficient voids so that incident wave energy is dissipated by turbulence within the structure's interstices. Figure 2-1 shows a typical quarrystone rubble-mound structure.

(b) When designing structures for the coastal environment, there is always some probability that design conditions will be exceeded during the structure's lifetime. Rubble structures may experience damage under such conditions and still maintain their ability to function. Rubble structures are often designed for the 10-percent wave height or the significant wave height (the average height of the highest 10 percent of the waves or the average height of the highest 33 percent of the waves, respectively) occurring during a storm with a given return period. At any instant in time during that storm, a range or distribution of wave heights prevails with occasional waves that exceed the 10-percent or significant height (about 18 percent of the waves in the distribution exceed the significant height). Consequently, rubble structures need not be designed to withstand the highest wave in the spectrum for the storm with a given return period.

(c) Information on potential sources of construction materials such as concrete aggregates and armor stone for rubble structures along with information on the quality of those materials is needed to select from among various structural alternatives. The location of the source relative to the construction site determines the cost of transportation. The weathering ability and durability of armor and underlayer stone and the chemical composition of concrete aggregates can have significant impact on the structural performance and service lifetime of a coastal structure. Information on the yield of potential quarries, the maximum size, and the size distribution of armor stone and underlayer stone a quarry will produce should be used to design rubble-mound structures that maximize the use of the quarry's production in the structure's cross section. A disadvantage of rubble structures is their relatively high construction costs and possibly the limited availability of suitable stone near many project sites. Also, if a distant quarry must be used, stone transportation costs may be high.

(d) When quarrystone heavy enough for the required armor is not available or when weight limits preclude transporting armor stone over public highways, precast concrete armor units may be an acceptable alternative. A wide variety of concrete armor unit shapes have been developed (SPM 1984, EM 1110-2-2904). Concrete armor units generally have improved stability characteristics that lead to comparable levels of stability with lighter, smaller units.

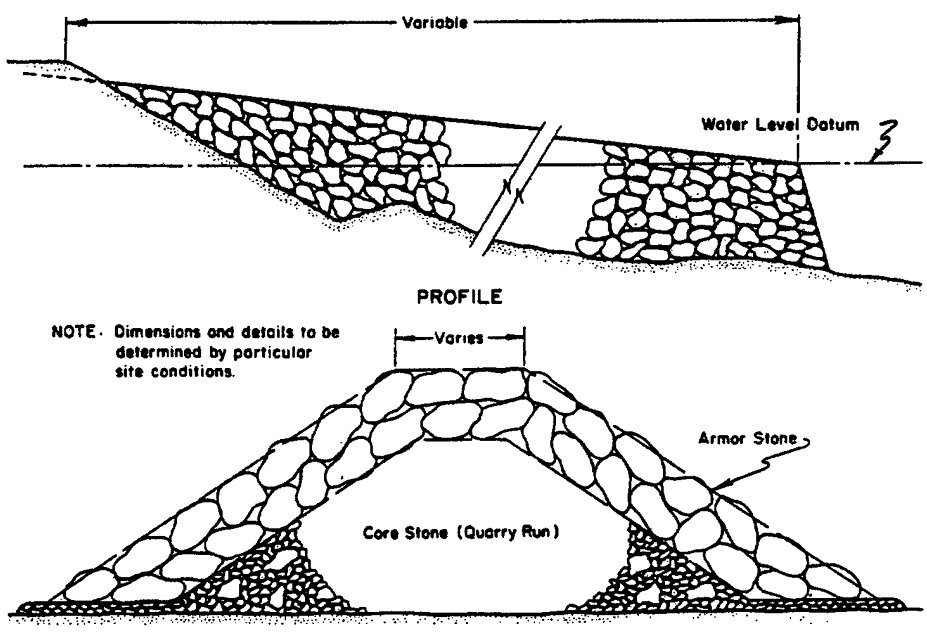
(2) Sheet-pile construction.

(a) Many functional groins, jetties, bulkheads, and in some cases, breakwaters and offshore sills have been built of sheet piling. Commonly, sheet piling used for shore protection has been timber, concrete, or steel. Sheet-pile structures usually have a relatively low initial cost since the volume of materials required is small, materials are readily available, and construction is usually faster than for comparable rubble structures. However, the service lifetime of these structures is often shorter, and therefore the life cycle cost may actually be higher. Sheet-pile structures are more rigid than rubble-mound structures and sustain damage if subjected to waves that exceed their design conditions. With the possible exception of good-quality concrete, the materials of which sheet pilings are made are less durable than stone in the marine environment. Deterioration and damage to sheet-pile structures often leads to a significant reduction in their ability to function properly.

(b) Sheet-pile structures reflect incident waves unless measures are taken to reduce their reflectivity. Often reflectivity is reduced by providing rubble along the structure. This rubble toe also serves as a scour blanket to prevent bottom scour. If wave reflections will not interfere with a structure's performance, sheet-pile structures may have an economic advantage.

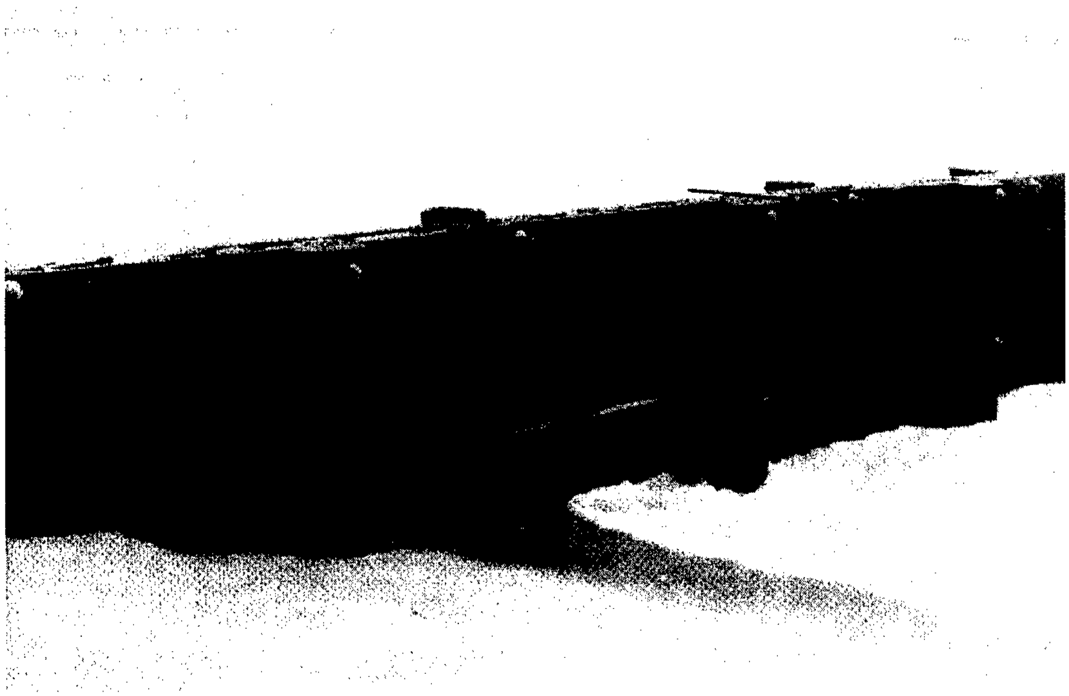
(c) Timber sheet-pile structures are often of ship-lap, tongue-and-groove, or Wakefield construction and are built of timber impregnated with creosote or some other preservative to slow deterioration and protect against marine borers. Overlapping timber sheet piles are usually jetted into the bottom, stiffened longitudinally by timber walers, and supported laterally by timber piles (Figure 2-2). Timber pile groins and bulkheads have been used extensively along ocean, Great Lakes, river, and

a. Westhampton Beach, New York (1972)

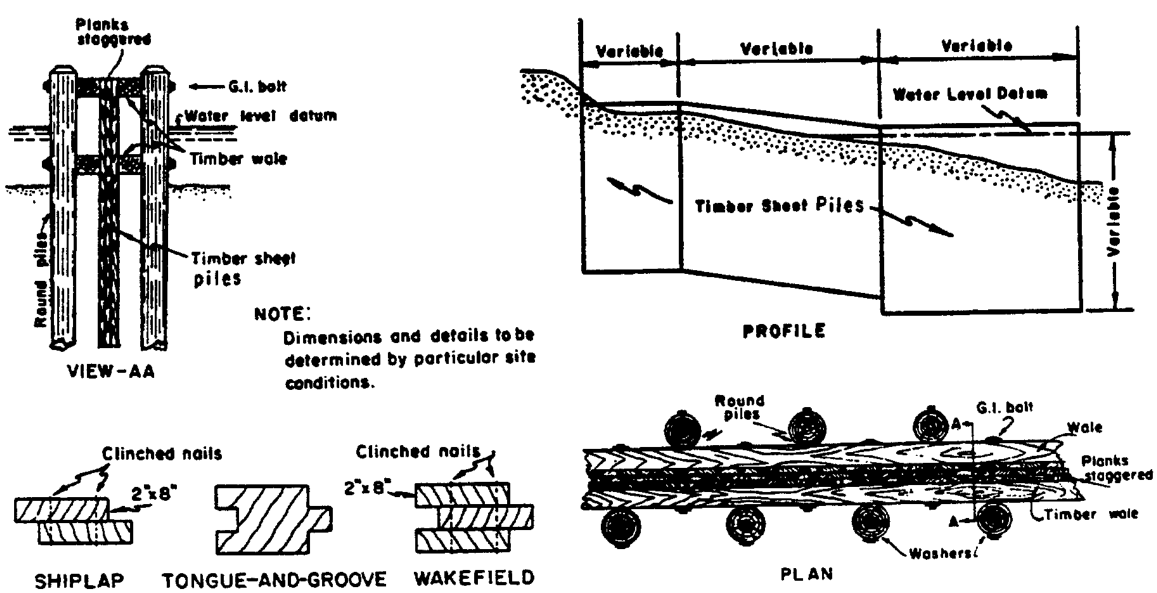


b. Cross section

Figure 2-1. Typical quarrystone rubble-mound groin



a. Wallops Island, Virginia (1964)



b. Cross section

Figure 2-2. Timber sheet-pile groin

estuary shorelines in the United States. Breakwaters and offshore sills built of timber sheet piles are less common.

(d) Properly designed concrete sheet-pile structures are more durable than structures built of other types of sheet piling. They are also usually more expensive. The dimensions of precast concrete sheet piles and the amount of reinforcing needed varies with the design. Lateral earth and wave forces usually establish critical design loads. Concrete sheet piles are designed with a key so that adjacent piles interlock. Longitudinal stiffness is usually provided by timber walers on both sides of the sheet piles fastened together with stainless steel bolts through holes precast into the piles or with a reinforced concrete cap (Figure 2-3). The concrete piles themselves usually provide lateral support or may be braced with tie-rods and piles. Groins, jetties, and bulkheads have all been built of concrete sheet piling.

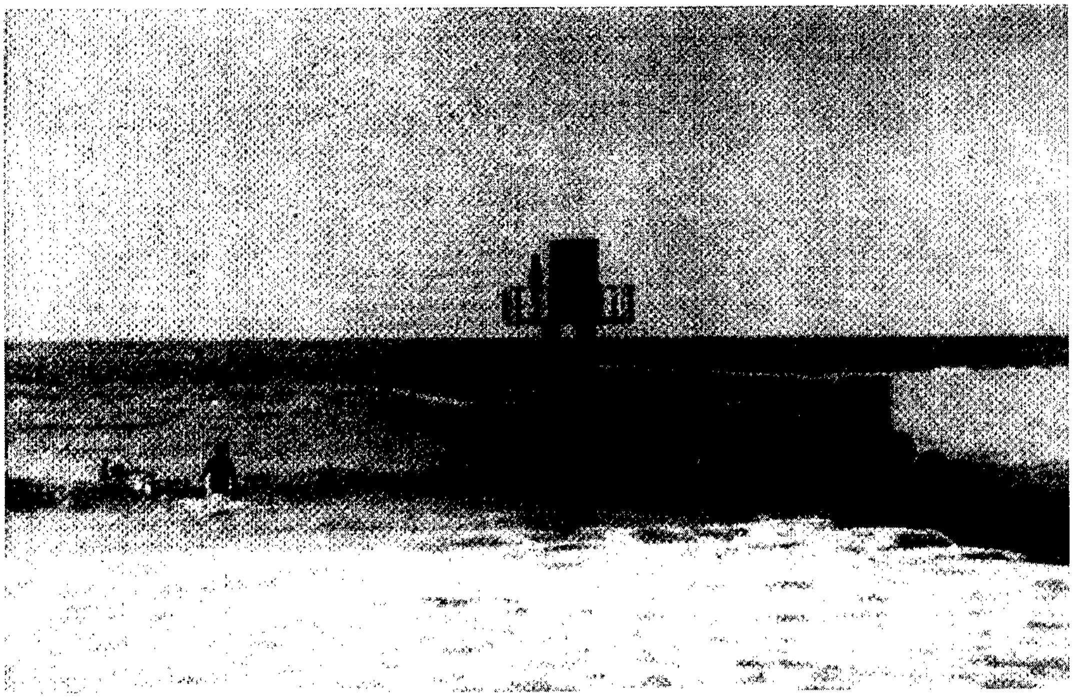
(e) Steel sheet piles are rolled structural shapes having various cross-sectional properties. The pile cross section, which may be straight, U-, or Z-shaped, has a channel along its edge that allows adjacent piles to interlock. Various section moduli are available to carry expected lateral earth and wave forces. Beach stabilization structures built of steel sheet piling are generally of two types: a single row of cantilevered piling with walers and often with adjacent piles to provide additional lateral support, and cellular structures. Structures built of a single row of piles are similar in design to the timber and concrete structures described. They are used primarily for bulkheads and low groins (Figures 2-4 and 2-5). Cellular structures are designed for large lateral loads. In plan, they consist of intersecting circular cells filled with earth, sand, or rubble and are then capped with rubble or concrete to contain the fill (Figure 2-6). Cellular sheet-pile structures have been used for both groins and offshore breakwaters, mostly in the Great Lakes.

(3) Other types of construction. Numerous other types of construction have been used for beach stabilization structures with varying degrees of success. For example, timber-crib structures have been used in the Great Lakes for breakwaters and jetties. These structures consist of a timber outer structure or crib into which rubble or stone is placed. This type of structure allows smaller stone to be used, which by itself would not normally be stable under wave attack. The timber crib allows the smaller stone to act as a unit. Gabions, wire baskets filled with stone, operate on the same principle but at a smaller scale (Figure 2-7). Gabions have been evaluated as low-cost shore protection, but are used primarily for stream bank or slope protection.

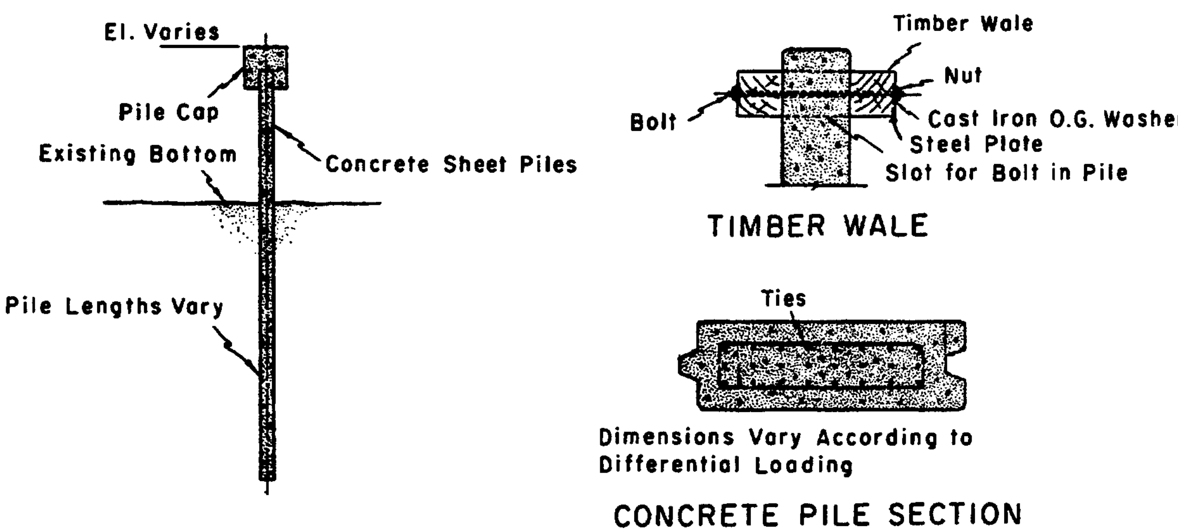
(4) Materials. Construction materials also impact on the effective service lifetime of beach stabilization structures. Timber structures that experience alternate wetting and drying, even those initially treated with wood preservatives, are subject to rotting whereas submerged portions are subject to marine borers when preservative protection deteriorates. Structural engineers should be consulted and involved in the selection of materials for beach stabilization structures. Determination of the best available material is dependent on many factors, such as expected project life, construction access, frequency, and accessibility of maintenance operations, and cost. These factors are considered in conjunction with the fact that these types of structures are located in severe, highly corrosive environments.

(a) Concrete. When reinforcement becomes exposed, especially in a saltwater environment, corrosion of the steel takes place causing cracking and spalling of the concrete. Methods of reducing this include: increasing concrete cover (concrete cover should be increased when designing structures for beach projects; proper consolidation is also critical to accomplishing this); use of epoxy-coated reinforcement, if necessary; and increasing the impermeability of the concrete. Retarding the ingress of chlorides and oxygen through the concrete is another method of reducing corrosion. This can be accomplished through the use of concrete mixes with low water/cement ratios. Type 2, sulfate-resistant cement should also be specified.

(b) Steel. Corrosion of steel members in coastal structures (which include piles, beams, channels, angles, tie-rods, and bolts) results in a loss of section that reduces the load-carrying capacity of the member. Selection of an appropriate protection system requires an assessment as to the feasibility (economically and logistically) of providing future maintenance. Coal tar epoxy is generally used in marine environments for protection of all members. Cathodic protection is another way to protect against corrosion; however, the cost of electricity and the replacement of sacrificial anodes increase operating costs. Aluminum and other metals may also react with seawater or soil. Abrasion of structural materials near the bottom by wave-agitated sand may also contribute to structural deterioration. In some cases, abrasion collars have been provided on structures at the sand line. Other conditions may prohibit the driving of steel piles, such as areas of hard, subsurface material and the existence of structures within the close proximity of driving operations.



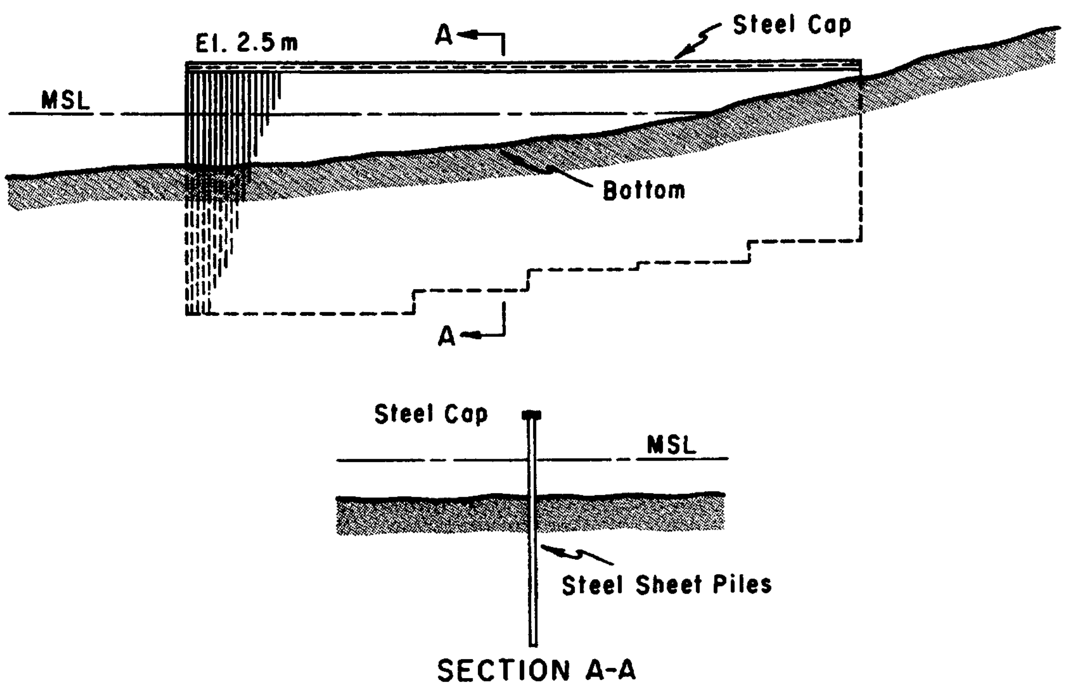
a. Doheny Beach State Park, California (October 1965)



b. Concrete pile section

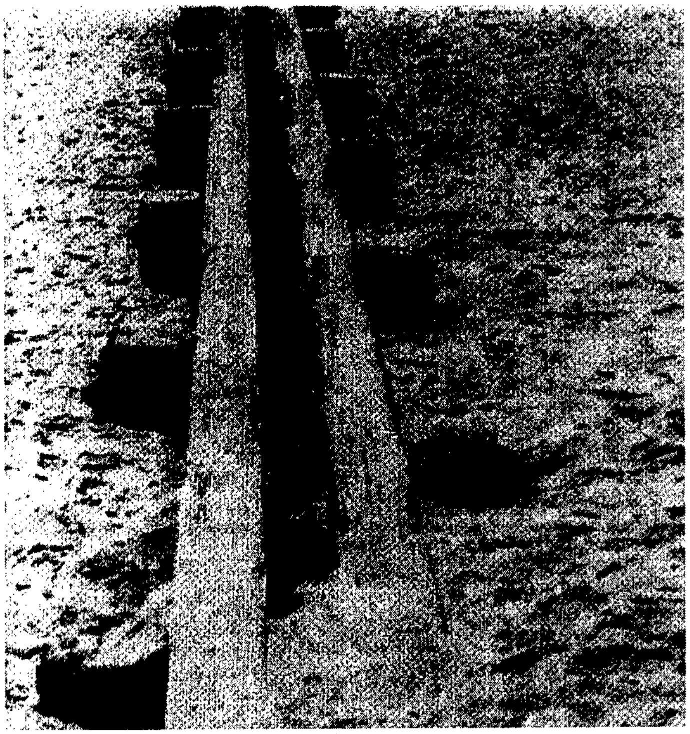
Figure 2-3. Cantieverted concrete sheet-pile structure with concrete cap (groin)

a. Newport Beach, California (March 1969)

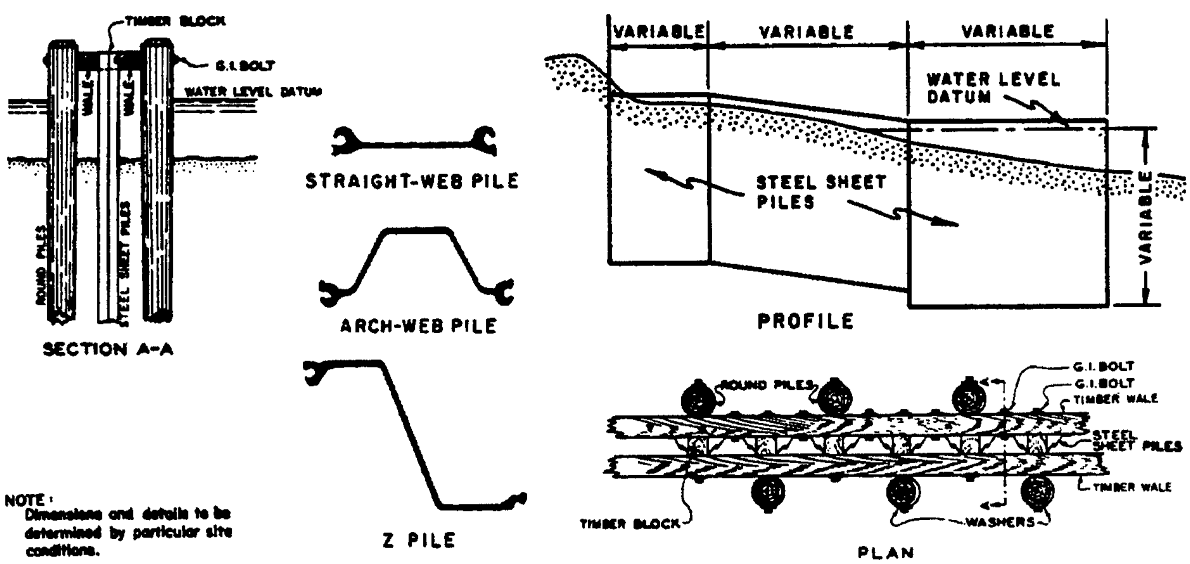


b. Cross section

Figure 2-4. Cantilevered steel sheet-pile structure with steel cap (groin)



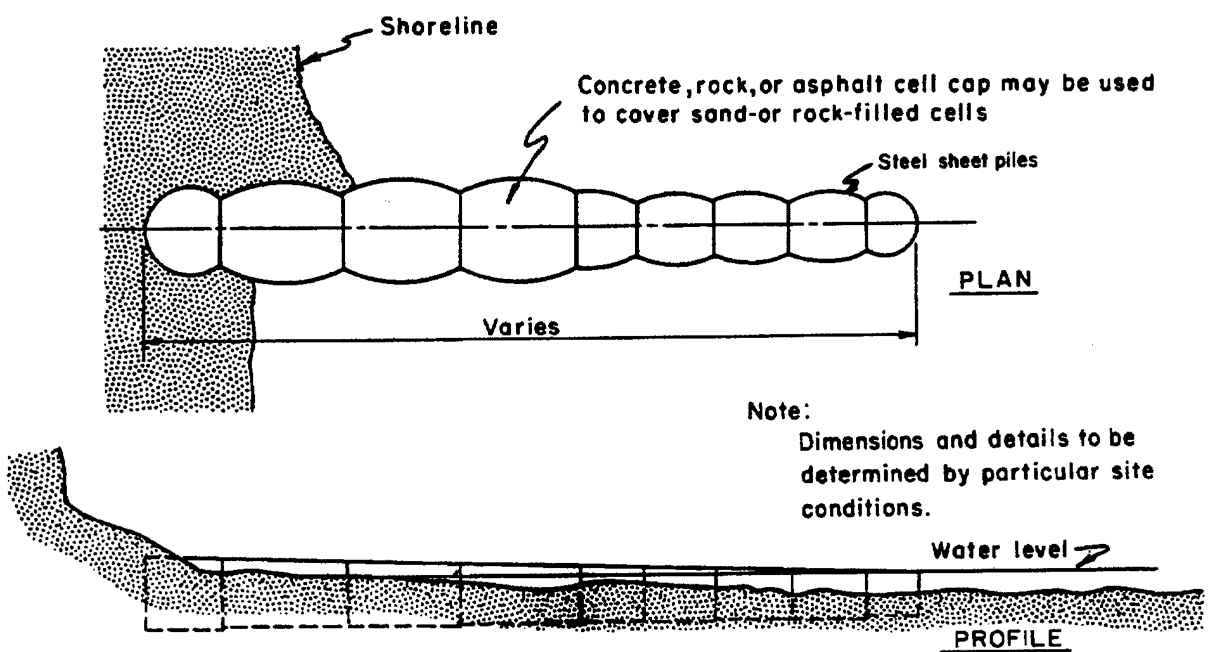
a. New Jersey (September 1962)



b. Cross section

Figure 2-5. Steel sheet-pile and timber wale structure (groin)

a. Presque Isle, Pennsylvania (October 1965)



b. Cross section

Figure 2-6. Cellular, steel sheet-pile structure (groin)

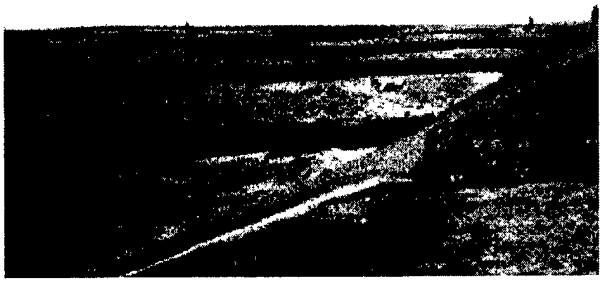


Figure 2-7. Gabion structure (revetments and groins)

e. Alternative beach stabilization methods.

(1) There are numerous proprietary beach erosion control and stabilization systems that function similar to groins, breakwaters, or submerged sills, but are of a unique geometry or type of construction. Most such structural systems are precast concrete units or flexible structures such as large sand-filled bags placed in various configurations on the beach or nearshore in shallow water. Most have undergone only limited field testing and many have never been field tested. Proponents of the various alternative schemes, usually the inventor or a vendor, often make unsubstantiated claims of success for their system. In fact, since they function either as groins, nearshore breakwaters, or perched beaches, they compete economically and functionally with traditional types of groin and breakwater construction such as rubble-mound and sheet-pile structures. The alternative structure systems, by themselves, do not increase the amount of sand available, but like their more traditional counterparts, redistribute available sand.

(2) Some of these structures have been evaluated under a program established by the Shoreline Erosion Control Demonstration Act, and their performance has been summarized by the Chief of Engineers in his report to Congress (Dunham et al. 1982). Field tests conducted under this program were all in sheltered US waters and not on the exposed ocean coast. Experience with most alternative beach stabilization systems on the open coast has been limited. In some cases, the results of experiments using open coast installations have not been reported because they have not been successful and, in some cases, successes have been selectively reported.

(3) One beach stabilization system, based on a different physical process, is beach-face dewatering. Under this system, a perforated drain pipe is installed beneath the beach face in the intertidal zone to lower the water table in the zone between low tide and the limit of wave runup at high tide on the beach (Figure 2-8). The lowered water table produces a ground-water hydraulic gradient in a direction opposite to that which normally prevails on a beach. This in turn results in the buildup of sand on the beach face. Presumably, sand in the water carried up the beach during wave uprush is not carried back offshore in the return surface flow, but rather, the altered ground-water gradient causes the surface flow to infiltrate into the sand, leaving the sand behind on the beach face. The result is an initial buildup of sand and stabilization of the beach face. Water collected by the perforated drain is carried to a collector pipe and then to a sump from which it is pumped back to the sea. The system thus requires drain and collector systems buried on the beach face and a sump and pumping system, which must be operated either continuously or periodically. Beach dewatering systems have been installed in Florida (Terchunian 1989), Namibia, and Denmark (Hanson 1986). Laboratory studies of beach dewatering systems have been conducted by Machemehl (1975) and Kawata and Tsuchiya (1986). Bruun (1989) and Parks (1989) also discuss beach dewatering.

(4) Evaluations of alternative beach stabilization systems should be based on their functional performance, their economics relative to traditional types of groin and breakwater construction, aesthetics, and their ability to be removed or modified if they do not function as expected or become aesthetically unacceptable. Since many systems are patented, they may also involve sole-source procurement or the payment of royalties to the inventor or licensee.

2-2. General Data Requirements for Design

a. Water levels.

(1) The range of possible water levels in the vicinity of a project is needed for both functional and structural design of beach stabilization structures. Prevailing water levels will determine where wave forces act on a structure and where the erosive action of waves will be felt on the beach profile. For example, during high-water levels, waves might attack the toe of a bluff that is normally above the active beach profile.

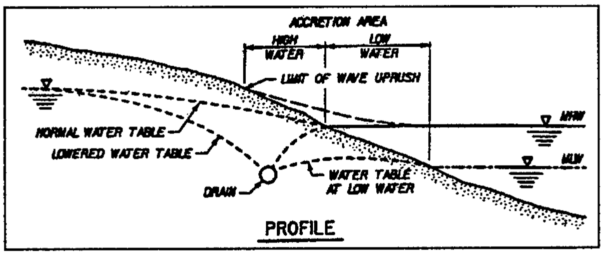


Figure 2-8. Beach dewatering system--lowered beach water table on beachface

(2) Many coastal structures extend across the surf zone so that different elements of the structure are subected to critical design conditions at different water levels. Thus, designs should not ordinarily be based on a single design water level, but rather on a range of reasonably possible water levels. For example, at low water the seaward end of a groin might experience breaking waves while more landward sections of the groin experience broken waves. At higher water levels, a more landward section of the groin might experience breaking waves, and the seaward end will experience nonbreaking waves. Sometimes the stability of a rubble structure depends critically on the water level at the toe of the structure since the stability coefficient depends on whether the waves are breaking or nonbreaking waves. The location on a structure where a wave of given height and period breaks depends on water depth and nearshore slope; hence, there will often be a critical water level where maximum wave effects (minimum structure stability or maximum forces) occur. Design calculations should recognize this factor, and a reasonable range of water depths should be investigated.

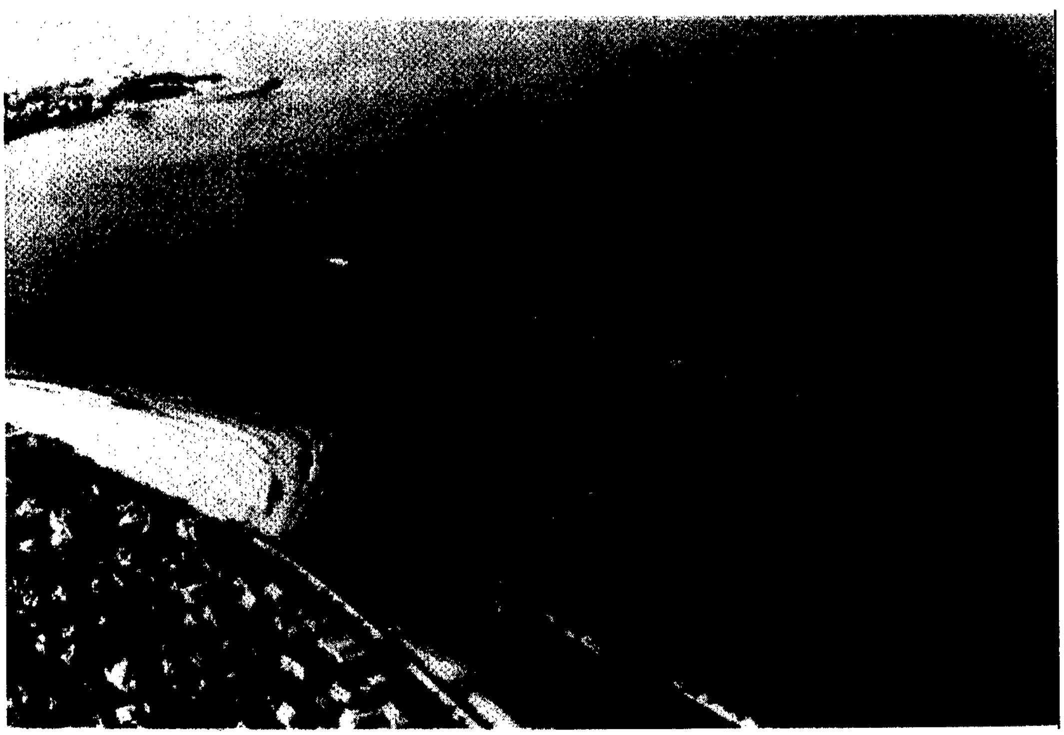
(3) Data on the range of water levels expected at a breakwater site are needed to determine the variation in a breakwater's distance from shore. During high-water levels, a breakwater will be farther from shore than during low-water levels. Some nearshore breakwaters have been observed to have significantly different low-water shorelines than high-water shorelines. For example, at Winthrop Beach, MA, a tombolo is exposed at low tide while only a salient is present at high tide (Figure 2-9). Wave conditions in the lee may be affected by prevailing water levels. Also, as water levels increase, freeboard is reduced, and wave overtopping of the breakwater may occur. Statistical data on water levels and the resulting breakwater freeboard establish the frequency of wave overtopping, a factor that influences the shape of the shoreline behind the structure. Frequent overtopping can prevent the formation of a tombolo and may also result in currents through the gaps in multiple breakwater systems. Surf zone width may also change the area where longshore transport occurs relative to the breakwater.

(4) Because water level changes are caused by astronomical tides, storm tides, and in the case of the Great Lakes, long-period hydrologic factors, water levels are usually described statistically. The frequency, or probability that a given water level will be equaled or exceeded, or its return period in years (the reciprocal of the probability of exceedence) is defined (Figure 2-10). Thus, for example, the water level that is exceeded on average once in 100 years (a probability of 1/100 = 0.01 of being exceeded in any 1 year) might be specified as a design water level. Significant deviations from predicted astronomical tidal levels will occur during storms because of meteorological tides (storm surges) caused by strong onshore winds and low atmospheric pressure. Consequently, design water levels for a structure may include a storm surge with a specified return period. The statistics of meteorological tides are usually based on recorded water levels at tide gaging sites or joint probability analysis of storm parameters and predicted surge heights.

(5) Water level data for coastal sites are often available from Corps of Engineers' General Design Memoranda for coastal sites where earlier studies have been conducted, Federal Emergency Management Agency (FEMA) flood insurance studies, or the National Oceanic and Atmospheric Administration's (NOAA's) National Ocean Service (NOS) for areas where NOAA operates tide gages. The location of NOAA's principal tide measuring stations along with the period of record are given in the annual NOAA "Tide Tables" publication (for example, see NOS 1986). Data on historical water levels of the Great Lakes and lake level statistics are available from NOS (1986) and from the US Army Engineer District (USAED), Detroit (for example, USAED 1986). Water level statistics for the US East Coast are given by Ebersole (1982). Water level statistics for predicted astronomical tides are also given by Harris (1981). This statistical compilation provides information on the fraction of time that water levels will be above a given level at a site (Figure 2-11).

(6) Studies by the National Academy of Sciences (Charney et al. 1979, Dean et al. 1987) and the Environmental Protection Agency (Hoffman 1984, Barth and Titus 1984) indicate that the rate at which sea level is rising may increase in many areas of the world as the possible result of a general global warming trend. Past rates of sea level rise (where sea level has been rising)

a. Low tide



b. High tide

Figure 2-9. Breakwater at Winthrop Beach, MA 1981) (Dally and Pope 1986)

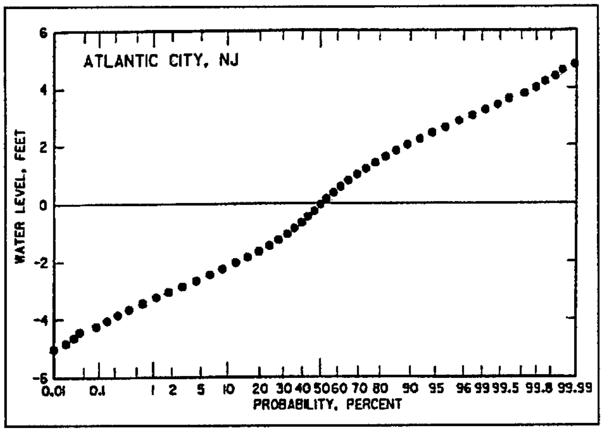


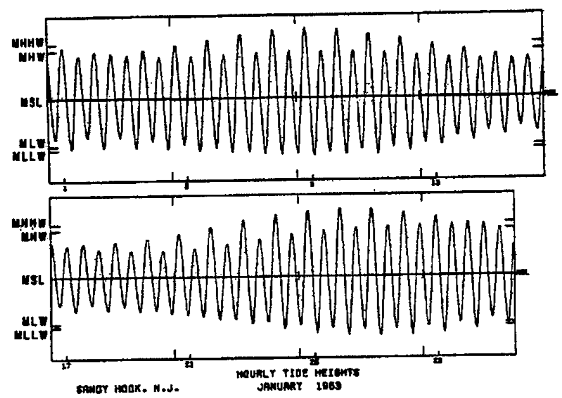
Figure 2-10. Statistical distribution of annual net longshore transport rates. (To convert feet into meters, multiply by 0.3048)

have been less than 1 foot (0.3048 meter)/century. The rate of relative sea level rise will vary with geographic location because it is influenced by local land subsidence or rebound. Data on local US experience with relative sea level change are summarized in Hicks (1973) and Hicks et al. (1983). Projection of past historic relative sea level change should be used in project design. Long-term erosion rates have been correlated with increases in local mean sea or lake level (Bruun 1962, Hands 1981). Procedures to calculate long-term erosion rates attributable to a rise in water level are given in Bruun (1961) and Weggel (1979). If the rate of relative sea level rise changes, the rate of erosion will likewise change. Prudence may require an allowance in a project design for the continuation over the project design life of an established significant long-term trend in relative sea level rise. Consideration must be given to the confidence band of the data the designer is using, the tolerance allowed in constructing the project, and whether it is more cost effective to include the allowance for the significant sea level rise in the initial construction or to plan for modification later, after the need for such is demonstrated.

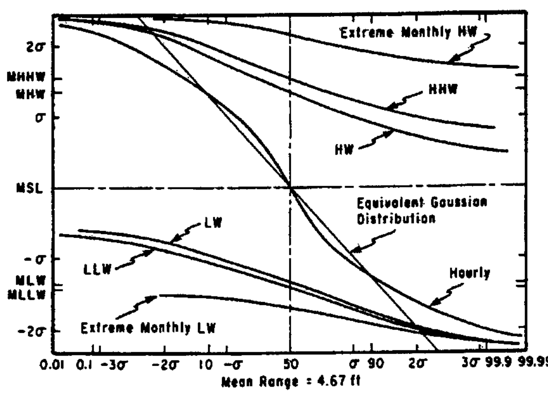
b. Waves.

(1) Wave data are needed for both structural and functional design of beach stabilization projects. Waves generally cause critical design forces on coastal structures. Waves also transport sediments onshore, offshore, and alongshore and therefore can transport sediments into and out of a project area as well as redistribute it within an area.

(2) Wave data required for structural design differ from data needed for functional design. For structural design, a characteristic wave height associated with a given frequency of occurrence or return period is usually needed. Thus, for example, the significant or root-mean-squared (rms) wave height that is exceeded on average once in 50 years or once in 100 years might be chosen for design. The largest probable wave for the given sea state and storm duration might then be selected for the structural design, or a lower wave in the spectrum (such as the 10-percent wave or the significant wave) might be used if a flexible structure such as a rubble-mound groin or breakwater is being designed. Ultimately, the selection of a design wave should be based on an evaluation of the



a. Hourly tide heights



b.Comparison of water levels

Figure 2-11. Statistics of predicted astronomical water levels (Harris 1981). (To convert feet into meters, multiply by 0.3048)

consequences of a structural failure, both the public safety and economic consequences. Structural design, therefore, focuses on the larger waves in the wave climate at a site since large waves generally result in critical design conditions.

(3) For functional design, a more complete wave data record is needed because sediment can move under even relatively small waves. The time series of wave height, period, and direction is needed to estimate the amount of sediment in transport alongshore. Net and gross transport rates are usually the summation of daily transport rates computed using Method 3 outlined in the SPM (1984). The SPM equation for estimating longshore transport rates requires knowledge of a characteristic wave height (usually the significant height), a characteristic wave period (usually the period of maximum energy density in the wave spectrum), and wave direction relative to the trend of the shoreline.

(4) For functional design of breakwaters, wave heights, periods, and directions are needed primarily to determine longshore sand transport rates. Incident wave heights, periods, and directions also determine wave conditions in the lee of a nearshore breakwater and establish the shape of the shoreline. The shoreline that evolves behind the structure depends on the range of wave heights and directions at the site and their seasonal variability.

(5) For groin design, wave height statistics and water levels are needed to determine the level of wave action to which various portions of a groin will be subjected. Because of its nearshore location, waves along the shoreward portion of the groin will be depth limited, i.e., maximum wave heights depend on water depth, wave period, and beach slope as given in Figure 2-12. Waves may or may not be depth limited at the seaward end of a groin depending on the prevailing water depth and on the height of incoming waves. Figure 2-12 can be used to determine the water depth seaward of which waves are no longer depth limited if the local height of the incoming waves is given as a function of water depth (Figure 2-13). For wave force and rubble-mound stability computations, design wave conditions with a given return period are usually specified, e.g., wave conditions with a return period of 20 or 50 years might be specified as the design wave height.

(6) Wave height statistics to determine design conditions will normally be based on hindcast wave data because a relatively long record is needed to confidently extrapolate the data. Wave gage records rarely cover a sufficient number of years to permit extrapolation. Corson and Tracy (1985) present extremal wave height estimates for 73 Phase II Stations of the Wave Information Study (WIS) Atlantic coast hindcasts. Also, Phase III WIS data for nearshore locations (Jensen 1983) can be plotted on extremal Type I (Gumbel) probability paper and extrapolated to longer return periods. Figure 2-14 is

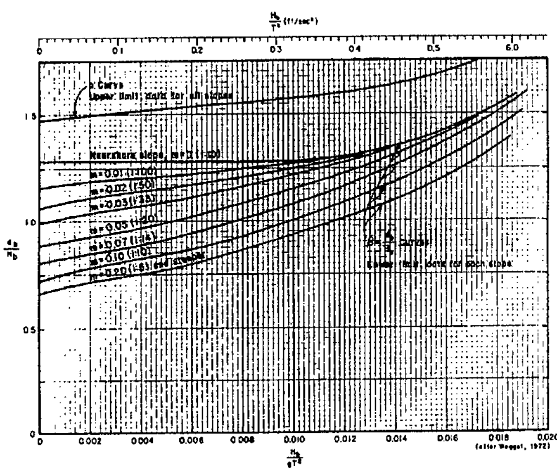


Figure 2-12. Water-depth-to-wave-height ratio at breaking as a function of wave steepness and beach slope (after Weggel 1972)

a plot of annual maximum wave heights ranked by height as a function of return period determined from the Weibull plotting position formula:

$$T_R = \frac{N + 1}{m} \qquad (2\text{-}1)$$

where

T_R = return period in years

N = number of years of record

m = rank of the given wave height (m = 1 for the largest annual wave height

m = 2 for the second largest, etc.)

(7) The prevailing wave direction will determine the shoreline orientation. The shoreline will move to orient itself more nearly parallel with incoming wave crests. If waves approach a beach from a predominant direction during one season, in time the shoreline will shift until it is parallel with the incoming waves of that season. When the direction of wave approach changes, the shoreline will eventually shift in response to the change if the wave conditions persist. For example, if the direction of

incoming waves changes for a period of time, the fillet in a compartment between two groins may shift from one groin to the other. The amount of sand in the groin

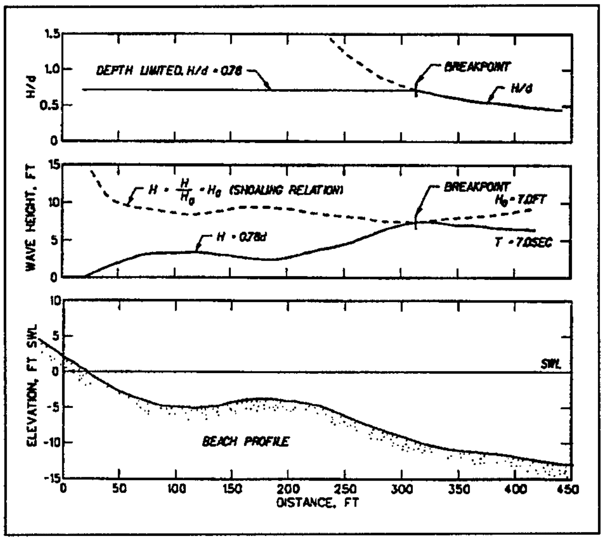


Figure 2-13. Wave height as a function of water depth and bathymetry, shoaling wave over irregular beach profile

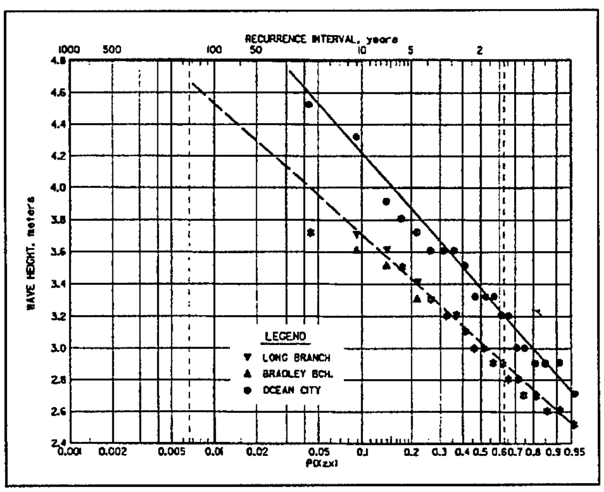


Figure 2-14. Annual maximum wave heights as a function of return period, Long Branch, Bradley Beach, and Ocean City, NJ

compartment is usually assumed to be conserved so that if the wave directions are known, the shoreline response can be determined once the profile shape is known. The best indicator of prevailing wave direction is the shoreline orientation at nearby groins.

(8) Application of wave and water level data to predicting onshore/offshore transport rates is not well developed, although in recent years several beach profile evolution models have been developed (Swart 1974, Kriebel 1982, Hughes 1983, Kriebel and Dean 1985). In addition, several models for beach profile and dune response to storms are available (Edelman 1968, Edelman 1972, Moore 1982, Vellinga 1983, Larson et al. 1990). Generally, the beach profile shape and its evolution depend on wave height, water level, wave-height-to-wave-length ratio (wave steepness), antecedent wave and beach profile conditions, and sediment characteristics such as mean grain size, grain size distribution, and grain shape. Wave conditions and water levels prevailing during both typical and extreme storms in a coastal area may be needed to evaluate the performance of a particular beach and dune profile and any associated beach stabilization structures. Additional guidance on water levels and wave heights for coastal design is provided in EM 1110-2-1412 and EM 1110-2-1414.

c. Longshore sand transport rates.

(1) Longshore transport is the most significant process for moving sediments in the coastal zone. Information on prevailing longshore sand transport rates is needed for the planning and design of all beach stabilization projects. The longshore sand transport rate, Q, is a measure of the rate at which littoral material moves alongshore in the surf zone from currents produced by obliquely breaking waves. These transport rates are needed to perform sediment budget calculations for an area, determine the amount of sand naturally available to fill groins or offshore breakwaters, determine whether beach fill is necessary for a project, and estimate how much sand will bypass a project to nourish downdrift beaches. Pre- and postproject sediment budgets should be developed for both the immediate project area and the adjacent shorelines.

(2) Longshore sand transport rates are usually specified as annual rates. The annual net transport rate is the net amount of sediment moving past a point on the beach in a year. Mathematically, it is given by:

$$Q_n = \frac{1}{T} \int_{t}^{t-T} Q(t)\, dt \qquad (2\text{-}2)$$

where

Q_n = net longshore sediment transport rate

T = time period over which the transport rate is averaged (usually 1 year)

t = time

$Q(t)$ = instantaneous longshore transport rate (positive or negative depending on whether transport is to the right or left for an observer looking seaward)

(3) The annual gross transport is the total amount of sediment moving past a point regardless of the direction in which it is moving. Mathematically, it is given by:

$$Q_g = \frac{1}{T} \int_t^{t-T} |Q(t)| dt \tag{2-3}$$

(4) The net and gross transport rates in terms of the positive and negative rates are given by:

$$Q_n = Q(+) - Q(-) \tag{2-4}$$

and

$$Q_g = Q(+) + Q(-) \tag{2-5}$$

where

$Q(+)$ = cumulative annual positive transport (total transport to the right per year for an observer looking seaward)

$Q(-)$ = cumulative annual negative transport (total annual transport to the left)

For the sign convention adopted, $Q(-)$, $Q(+)$, and Q_g are always positive, and Q_n may be either positive or negative.

(5) Therefore, the annual positive and negative transports are given by,

$$Q(+) = \frac{1}{2} (Q_g + Q_n) \tag{2-6}$$

and

$$Q(-) = \frac{1}{2} (Q_g - Q_n) \tag{2-7}$$

(6) The SPM (1984) suggests four ways of deriving longshore sand transport rates at a site. Method 1 recommends adoption of the best-known transport rate from a nearby site making appropriate adjustments if necessary to account for differences in exposure, sheltering, shoreline alignment, etc.

(7) Method 2 relies on documented sediment accumulations or shoreline changes in the vicinity of spits, inlets, or coastal structures. The volume of sediment accumulated in the time between two topographic/bathymetric surveys of the site is divided by the time between surveys to estimate the average rate of accumulation. Transport rates found in this way may approximate either the net or gross transport depending upon the process causing the accumulation. If based on accumulation at a spit, an estimate of net transport is obtained; if based on accumulation in an inlet, an estimate of gross transport is obtained. The basic principle involved in applying this method is to construct a simple sediment budget for a section of shoreline (or inlet) with the assumption that the influx and/or efflux of sediment is known at some location. At a spit, for example, the efflux at the distal end of the spit is assumed to be zero, and the net volume of sediment transported alongshore onto the spit accumulates there. (Changes in shoreline orientation along the spit and the resulting variations in longshore transport are generally ignored. This leads to some error.) For an inlet, sediment entering the inlet by longshore transport from either side of the inlet is assumed to be trapped, and the natural efflux of sediment from the inlet is zero. Thus, the gross longshore transport is estimated. Inlet dredging must be accounted for in determining the volume of sediment trapped. Any sediment naturally bypassing the inlet results in underestimating the gross transport.

(8) Method 3 is based on the assumption that the longshore transport rate, Q, depends on the longshore component of energy flux in the surf zone. The "Coastal Engineering Research Center (CERC) formula" (Equation 4-49, SPM 1984) for estimating the potential longshore transport rate is given by:

$$Q = \frac{K}{(\rho_s - \rho)\, g\, a'} P_{ls} \qquad (2\text{-}8)$$

where

K = dimensionless empirical coefficient
ρs = sediment density
ρ = water density
g = acceleration of gravity
a' = solids fraction of the in situ sediment deposit (1 - porosity)

$$P_{ls} = \frac{\rho g}{16} H_{sb}^2\, C_{gb} \sin(2\, \Theta_b) \qquad (2\text{-}9)$$

where

H_{sb} = nearshore breaking height of the significant wave
C_{gb} = wave group speed at breaking
Θ_b = angle breaking wave crest makes with the shoreline

In shallow water,

$$C_{gb} = \sqrt{g d_b} \qquad (2\text{-}10)$$

where d_b is the water depth at breaking, usually assumed to be linearly related to the breaking wave height as,

$$H_b = \gamma\, d_b \qquad (2\text{-}11)$$

where the breaking wave index, γ, is equal to 0.78.

(9) Equation 2-8 provides an estimate of the longshore transport rate in terms of breaking wave parameters. Wave data estimates may be obtained through Littoral Environment Observation (LEO) data (Schneider 1981) or by transforming waves inshore to breaking from an offshore source such as a wave gage or WIS data. The effect on a project of daily and seasonal variations in transport conditions can be studied when variations in wave conditions are known. For example, wave height, period, and direction data available from WIS wave hindcasts may be used to estimate a typical time series of longshore transport. The SPM (1984) provides a more detailed explanation of the equations and assumptions used in Method 3. Computation of longshore flux using LEO data is discussed in Walton (1980).

(10) Method 4 provides an empirical estimate of the annual gross longshore transport rate, which is also an upper bound to the annual net transport rate. A variation of the equation developed by Galvin (1972) is given by:

$$Q_g = 0.03636 \sqrt{g}\; H_b^{5/2} \qquad (2\text{-}12)$$

where

Q_g = annual gross transport at a site
g = acceleration of gravity
H_b = average annual breaker height at the site

The average breaker height can be obtained by averaging visual observations such as those obtained under the LEO Program, WIS, or gage data. Equation 2-12 is dimensionally consistent.

(11) Another approach for examining longshore transport develops a sediment budget based on estimates of inputs including bluff recession and stream sediment contributions. This method is commonly used along the Great Lakes and part of the Pacific coast, since Equation 2-8 can greatly overestimate transport in areas deficient of littoral material. The potential littoral transport rates *Q(+)* and *Q(-)* are determined from respective wave energy. The concept of littoral cells is applied; that is, a cell consisting of a self-contained stretch of coastline with its own sand sources, losses or sinks, and littoral drift connecting the two. Losses include offshore channels, canyons, sand mining, etc.

(12) Longshore transport rates may vary significantly from year to year, making it necessary to incorporate flexibility into the design of any shore protection project. For example, the net transport at a site might be in one direction one year and in the other direction another year. Gross transport rates exhibit similar variability with large gross rates occurring during particularly stormy years and lower gross rates in relatively calm years. Figure 2-15 illustrates the variability of annual net transport rates calculated from the WIS data for a site along the North Atlantic coast. This figure suggests that annual net longshore transport rates may be described by a Gaussian or normal probability distribution. The mean of the resulting distribution is the long-term average net longshore transport rate. The standard deviation of the distribution provides some measure of the annual variation

of the net longshore transport rate. The example distribution in Figure 2-15 shows that, on average, a year in which net transport is opposite to the long-term direction can be expected about once in 12.5 years for this site.

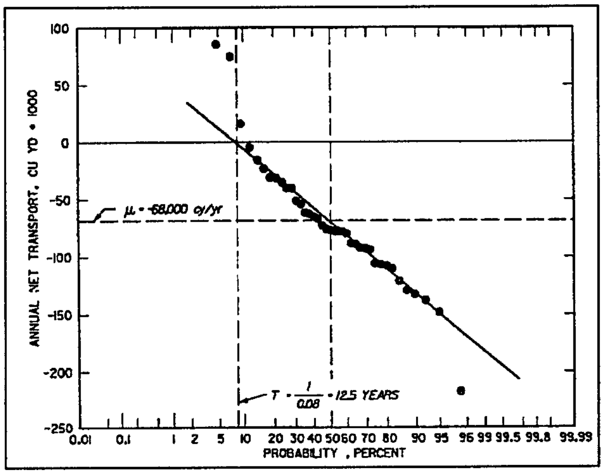
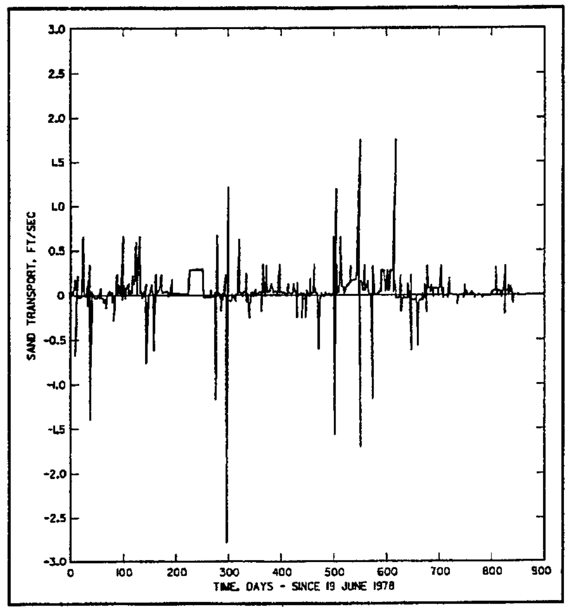


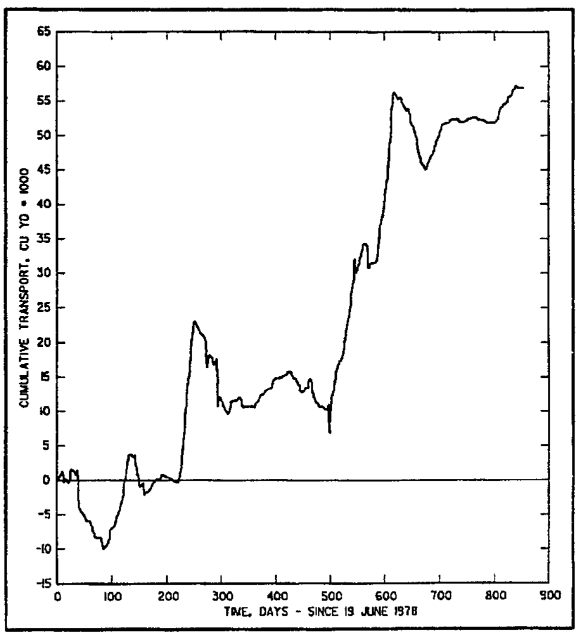
Figure 2-15. Statistical distribution of annual net longshore transportation rates. (To convert cubic yards to cubic meters, multiply by 0.76455)

(13) Longshore transport rates also vary seasonally. For example, along most reaches of the US Atlantic coast, net transport is southward during the winter months because of a relatively few intense "northeasters" that dominate the transport environment. These "northeasters" transport large volumes of sediment southward. During the late spring and summer months, net transport is northward because dominant waves are out of the southeast. Northward transport is usually smaller due to the lower wave heights generated during the spring and summer seasons. In response to the seasonal variations in transport direction, sand accumulation in the fillets adjacent to groins or behind nearshore breakwaters may move from one side to the other in response to prevailing transport conditions.

(14) Estimates of positive, negative, net, and gross longshore sand transport rates can be calculated from a wave climatology that includes wave heights, periods, and directions. Usually, the positive and negative (or the net and gross) transport rates will suffice for beach stabilization design. However, a time series of wave heights, periods, and directions permit the time series of longshore sand transport rates to be calculated. Figure 2-16(a) represents such a time series computed from daily visual wave observations. Figure 2-16(b), which is based on the data in Figure 2-16(a), is a plot of the cumulative amount



a. Time series



b. Cumulative longshore sand transport

Figure 2-16. Longshore sand transport, Slaughter Beach, Delaware

of sediment passing a point on the beach. With the development and improvement of computer models to simulate the evolution of shoreline changes near groins and breakwaters (Hanson and Kraus 1989).

d. Offshore bathymetry.

(1) Information on offshore bathymetry at a beach project site is needed for several purposes. If offshore structures or structures that extend seaward from the shore are being considered, bathymetric data are needed to establish the water depth at the site. This information will influence what type of shore protection is indicated, the wave and current forces to which they will be subjected, and the quantity of materials needed to build the structures. Offshore bathymetry is also important in the transformation of waves as they move from deep water toward shore. Wave refraction, shoaling, and diffraction by bathymetry alter local wave heights and directions. Locating potential sources of beach fill, such as offshore sand deposits and sand deposits in tidal inlets, also requires bathymetric surveys.

(2) Two bathymetric surveys of the same site spaced in time may be used to establish areas of accretion and erosion and to estimate erosion and accretion rates. The season when the two surveys were taken should be the same to distinguish long-term from seasonal changes. Bathymetric data can document the effect of structures on the offshore bathymetry and/or establish accretion/erosion patterns and rates in tidal inlets. Such accretion/erosion rates are needed to make sediment budget calculations and determine where and how much sand is available within an inlet for beach nourishment. More detailed analyses can also look at the patterns of erosion and deposition and the water depths in which these processes occur (Weggel 1983a).

(3) Approximate bathymetry for US coastal areas is given on US Geological Survey (USGS) 7.5-minute quadrangle topographic maps (quad sheets). However, bathymetry is continually changing, especially nearshore and in the vicinity of tidal inlets, capes, and river mouths, and these data may not be up-to-date. Naval Hydrographic Office charts also provide bathymetric data; however, they are intended primarily for navigation, and the bathymetry shown for shallow coastal areas away from established navigation channels may not be current. More recent and detailed bathymetric data may be available from the NOS in digital form or in the form of "boat sheets," raw data from which the bathymetry on USGS quad sheets is extracted. The preceding bathymetric data are often suitable for preliminary design or for wave transformation studies of areas distant from shore where bathymetric changes are less likely to occur. If up-to-date bathymetry is needed for project design or for documenting shoaling/erosion, it must usually be obtained during design. Special bathymetric surveys must be conducted if shore protection structures will extend offshore or if beach fill from offshore or inlet sources will be part of a project.

e. Shoreline changes. Measurements of shoreline changes are needed to establish short- and long-term erosion rates, determine typical and extreme seasonal movements of the shoreline, and determine the subaerial and subaqueous profile shape and its response to changes of wave conditions. Shoreline change data (both historical data and data obtained for a specific project's design) include profile surveys, aerial photographs, and other records documenting beach changes.

(1) Beach profiles.

(a) Periodic beach profile measurements that give the beach elevation along a line perpendicular to shore and extending offshore provide the most detailed information on shoreline changes; however, historical data may not be available for a given project site. Once a project is conceived and planning begins, a program of beach profile surveys should be initiated to acquire the needed data. Usually several years of such data are required. Profile data obtained during various seasons of the year are needed to establish normal and extreme seasonal shoreline movement and profile elevation changes. Storms usually occur more frequently during the fall or winter months when high, short-period waves result in "winter" or "storm profiles"; low, long-period, beach-building waves occur more frequently in summer resulting in "summer profiles" and wide beaches. In the Great Lakes, profiles respond to the seasonal rise and fall of the mean lake levels as well as to more long-period trends in water levels.

(b) If a groin is to serve as a template for the updrift postproject beach, the range of typical beach profile conditions at the site is needed to help establish the groin profile. The length of a groin is established by the expected beach profile adjacent to it and the desired location of the shoreline. The postproject profile is usually assumed to have a shape similar to the preproject profile; however, following construction, the profile on the updrift side of a groin will generally be steeper than the profile on the downdrift side (Figure 2-17). The difference in beach profile elevation between the updrift and downdrift sides of a groin will determine the lateral earth forces experienced by a sheet-pile groin and, since water depth

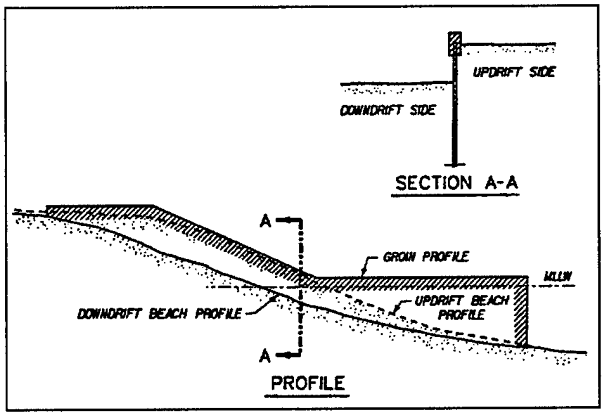


Figure 2-17. Groin profile showing differences in beach profile on updrift and downdrift sides

controls wave height in shallow water, the profile controls maximum lateral wave forces on a groin. Profile changes caused by scour adjacent to a groin must also be considered. Data on seasonal onshore-offshore profile movement are needed to determine the range of possible profile conditions on both sides of the groin. During periods when the groin is full and sand has built up against the updrift side, the profile determines how much sand will be transported over the groin on the beach face. A procedure for estimating shoreface transport rates over low groins and jetties is given by Weggel and Vitale (1985).

(c) Beach profiles can also provide data on the closure depth, the water depth beyond which there is no significant sediment movement (Weggel 1979, Hallermeier 1983). The closure depth plus the berm height gives an estimate of the beach area produced per unit volume of beach fill. For example, a closure depth of 27 feet with a berm height of 10 feet requires 27 + 10 = 37 cubic feet of sand to produce 1 square foot of beach. Even if beach fill is not part of a groin project, beach profiles and the closure depth are needed to compute sand volumes involved in beach alignment changes.

(d) Beach profiles at a nearshore breakwater project site are needed to determine the breakwater's location relative to the postproject shoreline and to estimate the volume of sand that will accumulate behind the breakwater. Except for in the immediate vicinity of the structure, profiles seaward of the breakwater can be assumed similar to preproject profiles. If beach fill is included in the project, the postproject profile will eventually be displaced seaward a distance approximately equal to the volume of fill per unit length of beach divided by the sum of the berm height and closure depth. The rate at which this seaward movement of the profile occurs is related to the rate at which the fill is distributed across the profile by wave action. This will occur more slowly for a nearshore breakwater project than for a beach fill without breakwaters. Beach profiles behind nearshore breakwaters will be steeper than preproject profiles. Preproject profiles will have to be adjusted using judgment in conjunction with any prototype data from similar breakwater sites to estimate how the postproject profiles will appear after construction.

(e) Offshore sills introduce a discontinuity into the nearshore beach profile. Preproject profiles can be used to estimate the postproject profile by shifting the preproject profile upward at the sill location. The amount of the shift depends on the height of the sill and on the time elapsed since placement of fill. The profile behind the sill will lower as the fill is eventually carried out of the area behind the sill. As this occurs, the profile will approach its preproject shape.

(2) Aerial photographs.

(a) Aerial photographs can provide quantitative information on shoreline location and a visual qualitative record on the location of underwater shoals, etc. Photogrammetric analysis can provide data on the elevation of the subaerial beach. Aerial photographs may be more readily available for a site than beach profile surveys since it is relatively simple and inexpensive to periodically photograph long stretches of coastline. Many states and Districts routinely obtain such photographs to provide historical records of shoreline changes.

(b) Shoreline location on an aerial photograph depends on the stage of the tide or water level (Great Lakes) and on the level of wave runup at the time the photograph was taken. Wave runup in turn depends on the height and period of the waves and on the beach slope. It is difficult to associate the water level visible on an aerial photograph with a particular datum. The photography could have been taken at low, mean, or high water level, or at any stage in between. Unless tied in stereoscopically with a vertical control datum, the datum will be approximate, especially for historical photographs where information on tidal stage at the time the picture was taken is not available. In addition, photographic distortions may be present that result in variations in scale from one portion of the photograph to another. Rectification of the photography will help to eliminate these distortions. If several sets of aerial photographs spanning several years are available, trends in the shoreline location can be determined. It is often easier to discern the bermline or a

debris line associated with high water on an aerial photograph instead of the waterline. The berm line or debris line will give more consistent information regarding beach erosion than will the shoreline. Figure 2-18 shows the

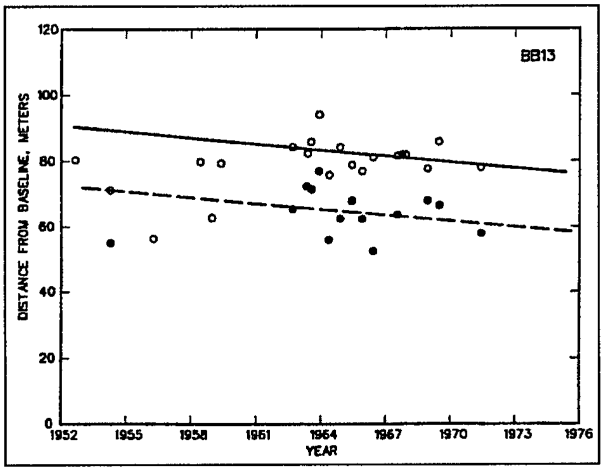


Figure 2-18. Bermline and high-water shoreline location as a function of time, data obtained from aerial photograph analysis (Bradley Beach, New Jersey)

bermline movement on a beach over a 20-year period. The bermline distance was measured relative to an arbitrary baseline located far enough landward so that it is not lost to beach erosion. The line on Figure 2-18 has been fit to the data and suggests slow but steady bermline recession and corresponding beach erosion. (The general beach profile shape has been assumed constant over the 20-year period of analysis.) The scatter of the data points about the trend line is a measure of both the seasonal fluctuations of the bermline (and shoreline) about the long-term trend and the errors involved in determining the bermline location on the photographs.

(3) Other documentation. Other data relating to beach changes include documentation of beach nourishment and sand mining. These might be in the form of tabulated data on volumes of sand placed on, or removed from, a beach or offshore area. For example, operations and maintenance dredging records used for contract payment might provide information on the quantity and location where sand was placed on a beach. Information on the exact distribution of sand along a beach might not be available; however, the quantity and general extent of its placement may be known and may explain observed beach changes found from aerial photograph or beach profile analyses.

f. Sediment budget.

(1) A sediment budget is a quantitative balance of the influx and efflux of sediment within a stretch of beach or other coastal area and the volumetric changes occurring on that stretch of beach. It expresses the conservation of sediment for a coastal cell with specific boundaries stating that the difference in the amount of sediment entering a coastal cell and the amount leaving will cause either erosion or accretion within the cell. If influx exceeds efflux, accretion occurs; if efflux excess influx, erosion occurs. An equation expressing this sediment balance is,

$$Q_{in} - Q_{out} = \frac{\Delta V}{\Delta t} \qquad (2\text{-}13)$$

where

Q_{in} = rate at which sediment is transported into the coastal cell from various sources

Q_{out} = rate at which sediment is transported out of the cell

ΔV = change in sediment volume within the cell

Δt = time period for which the sediment balance is being made

(2) There are generally several sediment sources and several sinks in any sediment budget analysis; Q_{in} and Q_{out} are each composed of several components. Sources of sediment may include longshore transport, cross-shore transport, wind-blown transport, bluff recession, rivers, and man-caused contributions of sand such as beach nourishment. Losses may be by longshore transport, offshore transport, wind-blown transport, transport down offshore canyons, transport into and trapping by tidal inlets, and man-caused losses due to dredging, sand mining, etc. In developing a sediment budget, most of these sources and sinks must be quantified, and the sediment balance equation solved for one unknown. A sediment budget may also balance sediment gains and losses between adjacent beach cells where sand lost from one cell becomes a sand gain for an adjacent cell. In this case, a system of simultaneous equations results (one equation for each cell) that can be solved for the several unknowns. Various assumptions may be made in formulating the equations and choosing what is assumed to be unknown. Typically, a sediment budget is developed for preproject conditions and calibrated using additional data if available. The effects of project construction may be tested by making various assumptions regarding the project's effect on longshore transport, offshore transport, etc. Often, sufficient data may not be available, or the data may not be sufficiently accurate to construct a

sediment budget. For example, small vertical errors in measuring offshore beach profiles can result in large errors in estimating sediment volumes. (A small error in elevation spread over large offshore areas results in large errors in volume.) In such cases, corrections to some components of the sediment budget may be necessary. Results from sediment budget analyses must always be carefully interpreted, and, whenever possible, the sensitivity of results to various assumptions should be tested. A detailed description of sediment budget analyses and their component elements is given in the SPM (1984, Chapter 4, Section VII), and example sediment budgets are given in Weggel and Clark (1983) and EM 1110-2-1502.

g. Other data requirements. Additional data needed for the design of a beach erosion control/stabilization project may include an inventory of existing structures, including their condition and effectiveness; geotechnical data; geophysical data; environmental and ecological data; and historical and/or archeological data.

(1) Existing structures. Data on existing structures might include an inventory of nearby structures and an analysis of their functional performance. The best indication of how a proposed structure will perform is the performance of a similar structure in a similar physical environment. An evaluation of how nearby groins, breakwaters, and sills are performing will provide an indication of how any proposed structures will perform. Also, if there are existing structures within a project area, a decision will have to be made to incorporate them into the project, simply abandon them, or demolish and remove them. This decision will require data on the structural condition and remaining useful life of the structures as well as data on their functional performance.

(2) Geotechnical data. Geotechnical data including the physical properties of underlying soils and their ability to support any proposed structures are required. Many coastal structures such as rubble-mound breakwaters and groins are gravity structures that significantly increase the overburden on underlying soils. Often beach sands are underlain with highly organic, compressible soils that originated in the lagoons behind barrier islands. As the barrier islands migrate landward, the lagoonal sediments appear on the seaward side of the islands. These strata consolidate under load and allow structures founded on them to settle. They may also fail in shear if the project requires that the overburden of sand be excavated to place the structure's foundation. Soil borings are necessary to locate any underlying strata and to obtain samples for testing. Similarly, pile-supported structures such as sheet-pile groins, etc., require data on underlying soil conditions for their design. EM 1110-2-1903, "Bearing Capacity of Soils," and EM 1110-2-2906, "Design of Pile Structures and Foundations," should be consulted for design guidance. In addition, Eckert and Callender (1987) address the geotechnical aspects of coastal structure design.

(3) Geophysical data. Geophysical data such as seismic reflection data can be used in conjunction with offshore core borings to locate and quantify offshore sand resources for beach nourishment. Relatively coarse, good quality sand obtained from offshore sources may provide a more economical alternative than nearshore sources for some beach restoration/stabilization projects.

(4) Environmental and ecological data. In addition to data on physical conditions at a site, baseline environmental data (preconstruction) and environmental monitoring (postconstruction) may be necessary, particularly if the project is expected to adversely impact the environment. Environmental data may include a baseline study of flora and fauna to identify potential environmental impacts that must be considered during the project's design. These baseline data can form a benchmark against which the results of a monitoring study can later be compared to assess the project's impact. A baseline study will identify the flora and fauna indigenous to the project area, identify and locate any endangered species, and provide data that can be used to identify any potentially adverse environmental impacts. Both subaqueous and subaerial communities and the anticipated effect of the beach erosion control/stabilization project on them need to be included. Environmental impacts may also occur at locations remote from the actual project, for example, at the sources of beach fill and construction materials. Beach-fill sand obtained from offshore requires dredging and thus affects bottom dwelling organisms. Environmental studies must be tailored to the specific needs of a given project. Additional guidance can be found in EM 1110-2-1204.

(5) Historical and archeological data. An archeological investigation might also be indicated if the proposed project is suspected to be near a historical site. This investigation would identify, map, and restrict access to historical or archeological areas endangered by the project.

2-3. Detached Breakwater and Groin Databases

a. Breakwater database. The US Army Engineer Waterways Experiment Station's CERC maintains a

database of detached breakwater projects in the United States and several other countries. The database includes information on the type and purpose of the breakwater, its date of construction, and various project dimensions. Information on the physical environment at the site is also provided along with a brief narrative description of the project's performance and any unique features.

b. Groin database. A similar database is being developed for groins; however, because of the large number of groin projects in the United States, a complete listing is not available. Only those projects having some unique feature are included.

Chapter 3
Groins

3-1. Objective

The objective of constructing a groin or groin system is to stabilize a stretch of beach against erosion where that erosion is due primarily to a net alongshore loss of sand. The beach may be either natural or artificially nourished. It may be intended for protection or for recreation; thus, groins can serve to protect an area or to maintain a wide recreational beach. Groins are narrow structures, often of rubble-mound or sheet-pile construction, that are usually built perpendicular to the shoreline. Groins may be used to build or widen a beach by trapping longshore drift, stabilize a beach that is subject to severe storms or to excessive seasonal shoreline recession by reducing the rate of sand loss by longshore transport, reduce the rate of longshore transport out of an area by locally reorienting the shoreline so that it is more nearly parallel with the predominant incoming wave crests, reduce longshore losses of sand from an area by compartmenting the beach, and prevent sedimentation or accretion in a downcoast area (i.e., inlet) by acting as a barrier to longshore transport.

3-2. Functional Design.

a. General. Functional design refers to determining whether groins can provide an acceptable solution to a beach erosion control problem. It involves determining the limits of a project area as well as the layout and dimensions of a groin or groin system to meet project objectives that may be to provide a protective beach or recreational beach with specified dimensions. It involves evaluating preproject conditions along a beach, estimating the effect of groin construction, and determining whether the amount of sand in longshore transport is sufficient to maintain project dimensions or whether it must be supplemented by beach fill. The frequency of nourishment must also be established.

b. Sediment budget. Functional design of groins requires knowledge of the sediment budget and longshore sand transport environment at a project site. Groins might be considered if the net sediment loss from a project area is by longshore transport, that is, if the amount of sand leaving the project area by longshore transport exceeds the amount entering. Groins may retain sand within a project area and reduce or stop sand loss to the downcoast area. Groin construction brings about changes in an area's sediment budget. These changes can be temporary or permanent depending on the type of groins, their dimensions, how permeable they are to sand, and whether beach fill is included in the project. The postproject sediment budget basically states that the rate of natural supply of sand entering the project area following groin construction, plus any beach nourishment, less the rate of sand loss from the area, equals the rate of accretion (or erosion) of sand in the project area. The estimated erosion rate will establish the required frequency for periodic nourishment. Note that the sediment budget for an area is dynamic, responding to daily and seasonal changes in waves, currents, etc. Therefore, a sediment budget based on long-term averages will not reflect these seasonal variations in transport conditions. Unfortunately, data are rarely available to do anything but a long-term sediment budget. A postproject sediment budget should also be developed for areas immediately downcoast and upcoast of a groin system to establish the extent of any sand deficit or shoaling problems caused by the groins. These sediment budgets can determine the extent of beach nourishment to include as part of a beach-fill project.

c. Types of groins.

(1) Groins, like beach stabilization structures in general, may be classified in several different ways. For example, they can be classified by the type of construction and by the materials of which they are built. Groins are routinely constructed of sheet piling, either as a single row of timber or steel piling with walers and adjacent piles for lateral support or as sand and stone-filled steel sheet-pile cells. At exposed ocean sites, groins are most often of rubble-mound construction because of the ability of rubble-mound structures to withstand wave conditions exceeding original design levels while continuing to function, their relatively low wave reflection coefficients, and the apparent ability of rubble-mound groins to reduce the chance of rip current formation. Sheet-pile groins are often provided with rubble-mound heads, that portion of the groin in deepest water and thus subjected to the highest waves, and they are often flanked with rubble to reduce reflections, minimize the formation of rip currents, and protect against scour with its resulting reduction in the groin's lateral structural stability.

(2) Groins are normally straight and perpendicular to the preproject shoreline; however, they are occasionally curved, hooked, or have a shore-parallel T-head at their seaward end. Occasionally, shore-parallel spurs are provided to shelter a stretch of beach or to reduce the possibility of offshore sand transport by rip currents. These latter refinements are generally not deemed effective in improving a groin's performance. They simply add to the

cost. The least amount of construction materials and the shortest groin length are obtained by a straight, shore-perpendicular structure. If T-heads are deemed necessary, shore-parallel, nearshore breakwaters should be considered as an alternative that eliminates the shore-connecting groin structure and thus reduces the volume of construction materials needed.

(3) Groins can be classified as either "long" or "short," depending on how far across the surf zone they extend. Groins that traverse the entire surf zone are considered "long," whereas those that extend only part way across the surf zone are considered "short." These terms are relative since the width of the surf zone varies with the prevailing wave height and beach slope. During periods of low waves, a groin might function as a "long" groin, whereas during storms it might be "short." Groins can also be classified as either "high" or "low," depending on how high their crest is relative to prevailing beach berm levels. "High" groins have crest elevations above the normal high-tide level and above the limit of wave runup on the beach. There is little wave energy transmitted over a high groin, and no sediment is transported over them on the beach face. "Low" groins have crest elevations below the normal high-tide level, and some sediment can be transported over the groin on the beach face. "Permeable" groins allow sediment to be transported through the structure; "impermeable" groins are sand tight. Most sheet-pile groins are impermeable. Some level of permeability, if desired, can be obtained with rubble-mound groins by adjusting the size of the stone and the cross-section design. Several patented precast concrete groin systems are designed to be permeable.

d. Siting.

(1) Length of shoreline to be protected is a consideration in siting the groin. The effect of a single groin on beach accretion and erosion extends some distance upcoast and downcoast from the groin. For a system of groins, the effect extends upcoast of the most updrift groin and downcoast of the most downdrift groin. The effect depends on groin length and probably extends some tens of groin lengths from the groin. The reach of shoreline stabilized by a groin system will depend on groin spacing, which in turn depends on groin length and prevailing longshore transport conditions. Groin length, in turn, is selected based on the width of the surf zone and on the amount of longshore transport the groin should impound. Protection will extend upcoast of the updrift groin; the distance it extends will depend on the wave environment. For areas where waves approach nearly perpendicular to shore, the distance updrift is greater than for areas where waves approach at a greater angle. (However, the time to impound sand is much greater owing to the lower longshore transport rates that prevail under nearly shore-parallel waves.) Similarly, the potential for significant erosion extends farther downcoast of the most downdrift groin. In areas where the direction of transport periodically reverses, the area of downcoast erosion may move from one end of the project to the other; however, because of the time required for erosion to occur, the severity of the erosion may not be as great under conditions of varying transport direction. The best way to establish the range of influence of a groin is to observe the effect of nearby groins or other longshore transport barriers on the beach. The beach alignment upcoast of a proposed groin should approximate the beach alignment upcoast of an existing transport barrier since the shoreline generally aligns itself parallel to incident wave crests characteristic of antecedent wave conditions. Thus if an existing groin or barrier is to be used to estimate the expected shoreline alignment, it should be observed over a period of time and during all seasons of the year to determine the range of possible alignments.

(2) Sand in the fillet updrift of a groin requires time to accumulate, particularly if the groin is filling by natural processes. Likewise, time is required for any downcoast erosion to occur. The amount of accumulation and erosion are greatest close to the groin and diminish with distance from the groin. The groin's effects propagate upcoast and downcoast from the groin. The rate of accumulation and erosion depends on the net rate of longshore transport. In areas where net longshore transport is high or in areas of nearly unidirectional transport, rates of accumulation and erosion will be high.

(3) Because of the potential for erosion along beaches downdrift from a groin system, a transition section composed of progressively shorter groins may be provided to prevent the formation of an area of severe erosion.

(4) Recent advances in the numerical computer simulation of shoreline evolution in the vicinity of coastal structures can be used to approximate the performance of a groin or groin system if the wave environment, including wave direction, is known (LeMéhauté and Soldate 1980, Perlin and Dean 1979, Kraus 1983, Hansen and Kraus 1989). Such models can be used to estimate the shoreline configuration as a function of time both upcoast and downcoast of a groin or groin system.

e. Groin length.

(1) Groins function by interrupting the longshore sand transport. Most longshore transport takes place in the surf zone near shore between the outermost breaking waves and the shoreline and also on the beach face below the limit of wave runup. Consequently, groin length should be established based on the expected surf zone width with the shoreline at its desired postconstruction location. Groins that initially extend beyond this point will impound more sand than desired, and the shoreline at the groin will accrete until sand eventually begins to pass around its seaward end. The sand fillet accumulated by the groin will then extend farther upcoast than desired (more sand will be impounded), and erosion will extend farther downcoast (a greater sand deficit will exist along downcoast beaches). Groins that do not extend across the entire surf zone will not intercept all of the longshore transport. Some sand will bypass the groin's outer end immediately following construction. This sand bypassing of the structure may be desirable to minimize erosion along downdrift beaches.

(2) The location of the surf zone varies with wave height and tidal stage; therefore, the relative groin length also changes with wave and tide conditions. Nearshore wave breaking occurs when a shoaling wave's height increases until the wave-height-to-water-depth ratio exceeds about 0.5 to 0.78; thus, higher incident waves break in deeper water farther from shore, the surf zone is wider, and the relative groin length is shorter. Similarly, at high tide incident waves of a given height will break closer to shore. Thus, at high tide the groin will be relatively longer.

(3) The SPM (1984) provides guidelines for estimating the trapping efficiency of groins (the fraction of the longshore transport trapped) depending on the water depth in which they terminate. These are estimates for the Atlantic coast with an average water depth at breaking of 1.8 meters. For long, high groins extending to -3.0 meters MLW (or Mean Lower Low Water, MLLW), 100 percent of the longshore transport is trapped. For high groins extending to between -1.2 and -3.0 meters MLW (or MLLW) or for low groins extending to less than -3.0 meters MLW (or MLLW), 75 percent of the longshore transport is trapped. For high groins extending to -1.2 meters MLW (or MLLW), 50 percent of the longshore transport is trapped. These are estimates of the equilibrium trapping/bypassing values that will prevail when the groin fillets are full.

f. Groin height and crest profile. Selection of a groin's height is based on several factors which will minimize the amount of construction materials used, control sand movement over the top of the groin, control wave reflections, and control the amount of sheltering from waves the groin provides to nearby downdrift beaches. Generally, a groin profile should have three sections: a high landward end with a horizontal crest at about the elevation of the existing or desired beach berm, a seaward sloping section that connects the high landward end with an outer or seaward section at about the slope of the beach face, and a seaward section generally with a lower elevation (Figure 3-1). However, most groins have been built with a constant crest elevation along their entire length, which causes increased offshore losses rather than allowing transport over the groin. The landward and sloping sections are intended to function as a beach template against which sand can accumulate on the updrift side of the groin. The groin profile is built to approximately the desired postproject beach profile. The seaward section is intended simply to prevent longshore sand movement in the surf zone. A higher seaward section shelters a portion of the downdrift beach and displaces any erosion problem farther downcoast. A lower seaward section will allow waves to carry some sediment over the structure and will reduce wave reflections from the groin. A significant amount of sand is transported on the beach face in the swash zone (Weggel and Vitale 1985); consequently, the amount of sand passing over a groin when it is full (overpassing) is determined by the elevations of the sloping and seaward sections.

g. Groin Spacing.

(1) The spacing of groins along a beach in a groin system is generally given in terms relative to the length of individual groins. The distance between groins is usually on the order of two to three groin lengths where groin length is specified as the distance from the beach berm crest to the groin's seaward end. Groin spacing should be selected by an analysis of the shoreline alignment that is expected to result following groin construction. Shoreline alignment is in turn a function of the wave and longshore transport environment at a site. It depends primarily on the prevailing direction of incident waves. When incident wave crests are nearly shore-parallel, a larger groin spacing can be used; when incident wave crests make a large angle with the shoreline, closer groin spacing is required. (When wave crests are nearly shore-parallel, longshore transport rates are small, and groins may not provide a satisfactory solution to an erosion problem.)

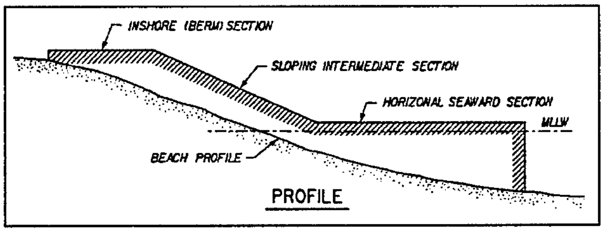


Figure 3-1. Typical groin profile showing inshore (berm) section, sloping intermediate section, and horizontal seaward section

For a specified direction of wave approach, optimum groin spacing can be determined by redistributing the sand within a groin compartment so that the shoreline is aligned parallel with the incoming waves. The quantity of sand contained within a groin compartment is assumed constant, and shoreline accretion at the downdrift end of the compartment is balanced by shoreline recession at the updrift end. If the project includes beach nourishment, the volume of beach-fill sand is included in the sediment balance. Similar calculations can be performed for various directions of wave approach to obtain insight into possible shoreline fluctuations due to seasonal changes in wave conditions. Details of computing the sediment balance within a groin compartment are summarized in the SPM (1984, Chapter VI, Section 3).

(2) When wave direction and transport rates are variable, the shoreline alignment near groins will also vary. Numerical computer models of shoreline response to groin construction in an environment of changing transport directions and rates can provide insight into how the shoreline will behave and the range of possible shoreline configurations that will result. Different groin spacings can be investigated for a given wave environment and the groin spacing that provides an optimum shoreline response selected.

h. Permeability.

(1) General.

(a) Permeability refers to the transport of sand through a groin; a permeable groin is one that will allow some sand to pass through it. Usually, sheet-pile groins are impermeable while rubble-mound groins will have some degree of permeability unless special precautions are taken to ensure that the groin is sand tight. Permeability may be desirable if some sand is to be bypassed to downdrift areas. There are no quantitative guidelines for determining the permeability to sand of a given groin geometry. Low rubble-mound groins have been used as terminal structures that allow controlled sand losses from a beach erosion control project to preclude erosion along adjacent beaches. The permeability of rubble-mound structures can be adjusted by adding or removing stone and by raising or lowering their crest elevation where it intersects the shoreface. A "vertical" barrier of geotextile fabric through the interior of the structure can reduce sand passage. However, this is a trial and error procedure, and actual permeability varies with water level and wave conditions.

(b) Several patented precast concrete groin systems are permeable, and some allow their sand bypassing to be adjusted. However, experience with these systems has been too limited to quantitatively predict their sand bypassing ability.

(2) Void sealing to reduce permeability.

(a) Occasionally rubble-mound terminal groins, jetties, or breakwaters are too permeable and allow sand and/or wave energy to pass through them. For example, a terminal groin may allow too much sand to leave a beach-fill project area; a jetty may allow sand to move through it from an adjacent beach into a navigation channel, or the voids in a breakwater may allow wave energy to be transmitted through it. Occasionally, voids exist due to design or construction deficiencies, but most often, voids develop or open in rubble-mound structures due to the loss of core stone resulting from storm wave action or due to structural settlement. Thus, many older structures may not function as intended because of an increase in their permeability.

(b) If the function of a structure is seriously impaired by its permeability, steps to seal the voids may be economically justified. The quantity of sand passing through the structure and the cost of dealing with it determines if void sealing is warranted. The first step is to determine whether sand is in fact passing through the structure or whether it is passing over or around the structure. This problem can often be identified by a study where dye is injected into the water updrift of a structure and signs of the dye are sought downdrift of the structure. Wave setup on one side of the structure creates a hydraulic gradient that causes a flow that in turn carries sand through the structure. If permeability is a problem, the dye appears downdrift within minutes of its updrift injection.

(c) Sealing voids in rubble-mound groins and jetties is discussed by Denes et al. (1990). Considerations include evaluation of materials used to seal voids, evaluation of how the sealant is to be installed, environmental impacts

of introducing sealant materials into the marine environment, and the long-term durability of the sealants. Void sealants include grouts, stiff aggregate-containing cements, and asphalt. Denes et al. (1990) investigated two cementious mixtures, a sodium silicate-cement mixture, a sodium silicate-diacetin mixture, and a sand-asphalt mixture. The cementious mixtures and the sand-asphalt mixture always hardened well whereas some problems were experienced with gelling of the sodium silicate mixtures and with their subsequent erosion and deterioration. To ensure a successful sealing project, Denes et al. (1990) recommend that a reliable, experienced contractor be employed; there be thorough inspection of the work while it is in progress to ensure that the structure is being adequately sealed; the job be evaluated while sealing progresses so that adjustments can be made as needed; and proper attention be given to spacing the injection boreholes to ensure an adequate distribution of the sealant. Rosati and Denes (1990) discuss a field evaluation of the rehabilitation of the south jetty at Port Everglades, Florida.

i. Shoreline orientation and its effect on longshore transport. Groin construction will result in the shoreline reorienting itself more nearly parallel with the prevailing incident wave crests. Following groin construction, the general shoreline alignment will be different than it was before construction. Net longshore sand transport rates along the reoriented shoreline will be lower because the angle between the average incoming wave crests and the new shoreline will be smaller. In other words, the shoreline will align itself so that positive and negative transport rates are more nearly balanced, thus yielding a lower net transport. If a time series of wave data are available, such as WIS hindcasts, the reduction in net transport can be estimated by calculating new transport rates for both the original shoreline and for the reoriented shoreline.

j. Terminal groins.

(1) The ends of beach nourishment/beach stabilization projects, where the project area abuts an adjacent inlet or a beach that is outside of the project area, require special attention. Significant amounts of sand can be lost from the project along with the associated economic benefits, or erosion can occur along sand-starved downdrift beaches. Terminal groins are constructed at the ends of beach nourishment projects to contain sand within the project area or to control the rate at which sand is lost from the project area by longshore transport. At inlets, sand lost from a beach nourishment project not only reduces the beach nourishment benefits, but it may also cause sedimentation and associated navigation problems within the inlet; consequently, a sand-tight terminal groin is necessary. Where nourishment projects abut beach areas, terminal groins that allow some sand bypassing may be needed to preclude erosion along adjacent beaches.

(2) Sand-tight terminal groins must be impermeable and are usually high and long in order to prevent sand from being carried through, over, or around them. Sand-tight rubble-mound terminal groins have an impermeable core usually of small, quarry-run stone or, in some cases, a sheet-pile cut-off wall. It is important to ensure that the design and subsequent construction assure a sand-tight groin since sealing the voids of an existing rubble-mound structure is expensive.

(3) Terminal groins designed to permit some sand bypassing are usually low, short, and permeable to sand. The amount of bypassing a given groin will allow is difficult to estimate; however, some guidance on transport over low groins and jetties is given in Weggel and Vitale (1985). Transport around the end of a groin can be estimated knowing the groin's length, the wave and longshore transport environment, and the cross-shore distribution of longshore transport. Hanson and Kraus (1989) discuss assumptions regarding bypassing around groins as related to the numerical model GENESIS. In general, longshore transport extends from the beach seaward to a water depth about 1.6 times the breaking depth of the transformed significant wave (Hallermeier 1983).

k. Groin system transitions.

(1) At the end of beach stabilization projects that employ groins and where the potential exists to erode downdrift beaches, a transition reach is often needed to go from the reach stabilized by groins to the adjacent unstabilized reach. The length of the groins at the end of the project is gradually decreased to form a transition from the project's typical groins to the adjacent beach (Figure 3-2). Generally, the groin shortening is effected along a line converging to the shore from the last full-length groin, making an angle of about 6 degrees with the natural shore alignment (Bruun 1952; USAED, Wilmington). The length of a groin is defined here as the distance from the bermline to the seaward end of the groin. The spacing between groins in the transition reach is also decreased to maintain a constant spacing-to-groin length ratio, *R*. The length of the first groin in the transition section is given by,

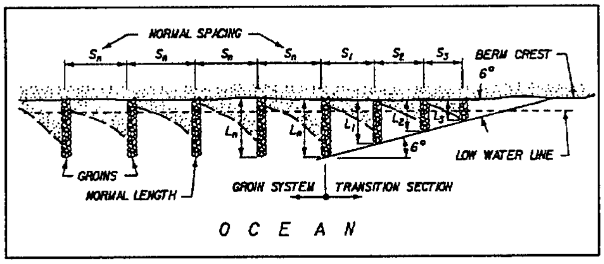


Figure 3-2. Transition section between groin field and beach not stabilized by groins

$$L_1 = \left[\frac{1 - \left(\frac{R}{2} \right) \tan 6^\circ}{1 + \left(\frac{R}{2} \right) \tan 6^\circ} \right] L_n \tag{3-1}$$

where

L_1 = length of the first groin in the transition
R = ratio of groin spacing to groin length in the groin field
L_n = length of the groins in the groin field

The spacing between the last groin in the groin field and the first groin in the transition section is given by:

$$S_1 = \left[\frac{R}{1 + \left(\frac{R}{2} \right) \tan 6^\circ} \right] L_n \tag{3-2}$$

where

S_1 = spacing between the groins

(2) These equations can be used recursively to calculate the length of each succeeding shorter groin in the transition and its distance alongshore from the preceding groin. Thus $L_2 = C_0 L_1$, $L_2 = C_0 L_2$, etc. Also, $S_2 = C_1 L_1$, $S_3 = C_1 L_2$, etc., where C_o and C_1 are given by the terms in brackets in Equations 3-1 and 3-2, respectively. C_o and C_1 are each constant for a given R. The shortest groin in the transition should extend seaward to at least the mean lower low water line. Groin system transitions can also be investigated using the numerical model GENESIS (Hanson and Kraus 1989).

l. Design to meet functional objectives. The functional design of groins is discussed in detail in the SPM (1984), Chapter 5, Section VI. Several rules of groin design are repeated here.

(1) Rule 1: Groins can be used only to interrupt longshore transport. Groins are ineffective in preventing the loss of sand by offshore transport. The normal onshore-offshore transport of sand is essentially unchanged by the presence of groins. Longshore transport, however, is trapped by groins until the shoreline builds seaward to the point where sand can move around the groin's end, or the groin's crest elevation is such that sand can move over it during periods of high water.

(2) Rule 2: The beach adjustment near groins depends on the magnitude and direction of the longshore transport. Groins reorient the shoreline so that it is more nearly parallel with the prevailing incoming wave crests. If the direction of the incoming waves changes, the shoreline will move to reorient itself parallel with the new wave direction. The shoreline thus reflects the wave conditions that prevailed for some time prior to the time when the shoreline was observed. For example, if transport is to the south, the beach will build up against the northerly side of a groin; if transport is to the north, the shoreline will shift so that the buildup is against the southerly side of the groin.

(3) Rule 3: The groin-induced accumulation of longshore drift on the foreshore modifies the beach profile, which then tries to reestablish its natural shape. The beach profile along the updrift side of a groin will be steeper than the profile along the downdrift side. At the seaward end of the groin, the updrift profile elevation and the downdrift profile elevation must be essentially the same and, since the distance from the seaward end of the groin to the beach berm along the updrift profile is shorter, the average slope along the updrift profile must be steeper than the average slope along the downdrift profile.

(4) Rule 4: Water pushed by waves into a groin compartment sometimes returns offshore in the form of rip currents along the sides of groins. Since groins cannot prevent offshore losses, rip currents induced by groins often carry large quantities of sand seaward. There are three mechanisms (Dean 1978) that can cause rip currents to develop adjacent to groins: the groin deflects the shore-parallel longshore current seaward; wave setup adjacent to a groin causes an increase in the mean water level there while the portion of the beach sheltered by the updrift groin has lower waves, resulting in a circulation

cell within the groin compartment that flows seaward along the updrift groin; and differential waves setup along the shoreline between two groins when waves approaching perpendicular to the beach cause two circulation cells with rip currents flowing seaward along each groin.

(5) Rule 5: The percentage of the longshore transport that bypasses a groin depends on groin dimensions, fillet dimensions, water level, and wave climate. Sand passes around the ends of relatively short groins, i.e., groins that do not extend beyond the seaward end of the normal surf zone. Sand passes through rubble-mound groins having large voids that make them permeable. Sand in suspension passes over low groins. Sand will also pass over a groin on the beachface between the water line and the limit of wave uprush if the beachface is above the groin's crest elevation.

(6) Rule 6: The longshore drift that is collected in the updrift fillet is prevented from reaching the downdrift area, where the sand balance is upset. Sand trapped and retained on the updrift side of a groin is sand that would normally nourish the downdrift beach. Preventing this sand from reaching the downdrift beach causes a sand deficit there.

(7) Rule 7: In the absence of other criteria or if the spacing determined by the shoreline analysis appears to be unreasonable, the spacing between groins should equal two to three times the groin length as measured from the berm crest to the groin's seaward end.

(a) Spacing between groins should be determined by a shoreline orientation analysis. The shoreline between groins is determined by the predominant direction of wave approach. As numerical models evolve, groin spacing will be determined by the computed shoreline response to a simulated wave and long-shore transport environment deemed typical of the groin site. In the absence of such a numerical simulation, the "rule of thumb" spacing given by Rule 7 should be used.

(b) Dimensional analysis. A dimensional analysis of the variables important in groin design can provide insight into the factors governing the functional design of groins. Details on dimensional analysis and an example application can be found in Appendix C.

3-3. Structural Design.

a. Loading

(1) Wave forces.

(a) Because groins are oriented nearly perpendicular to the shoreline, waves propagate along the groin's axis so that their crests almost make a 90-degree angle with the groin. For sheet-pile groins, lateral wave forces arise because a wave crest acts on one side of the groin whereas a lower water level acts on the other, e.g., either the still-water level or a wave trough. For directions of wave approach that make a small angle with the groin axis, Mach-stem wave reflection occurs (Figure 3-3). The incoming wave crest aligns itself perpendicular to the groin's axis, and the resulting wave height acting on the groin is higher than, but not twice as high as, the incoming wave (Figures 3-4 and 3-5). Wave heights on the leeward side of the groin may be lower. However, the groin should be designed for waves approaching from either direction. Wave loading on vertical sheet-pile groins and jetties is discussed in Weggel (1981). The loading procedure was verified in the laboratory by Hanson (1982) and is based on the Miche-Rundgren nonbreaking wave force diagrams in the SPM (1984, Chapter 7, Section 2). The force is distributed along the structure in proportion to the wave profile, and the wave profile is that of a conoidal wave. Figure 3-5 shows the reflection coefficient, and Figure 3-6 gives an example wave loading diagram. The maximum lateral force acts over only a portion of the structure at one time (at the location of the wave crest), and forces are distributed longitudinally along the groin by the walers.

(b) Most rubble-mound groins are designed with quarrystone armor heavy enough to be stable under a selected design wave height. A typical rubble-mound groin cross section is shown in Figure 3-7. Stone in the first underlayer is selected to be large enough so it will not fit through the voids of the armor layer; stone in the second underlayer will not fit through the voids of the first underlayer, etc. This criterion is met if the first underlayer weighs $W/10$ where W is the median weight of the armor stone. This criterion assumes that the stone in the underlayers has approximately the same unit weight as the armor stone. By this criterion, the second underlayer

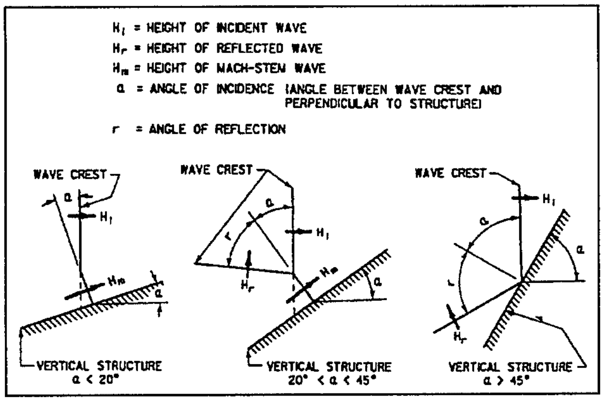


Figure 3-3. Reflection patterns of a solitary wave, oblique angle of incidence (Perroud 1957)

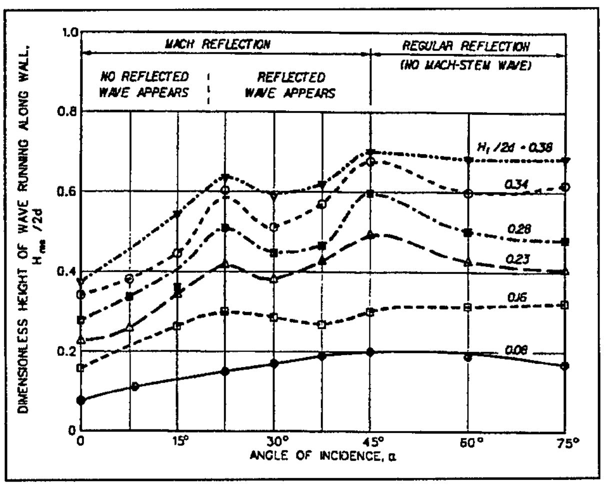


Figure 3-4. Oblique reflection of a solitary wave, Mach-stem reflection (Perroid 1957)

stone should weigh approximately $W/100$. The relationship between armor unit weight and design wave height is the same as that for jetties and breakwaters. More detailed information regarding the design of rubble-mound structures including groins is given in EM 1110-2-2904 and the SPM (1984, Chapter 7, Section III).

(c) Sheet-pile groins are often provided with rubble toe protection that serves as a scour blanket to prevent undermining and thereby a reduction in lateral stability. The stone weight needed for stable toe protection can be determined from EM 1110-2-1614 and the SPM (1984).

(2) Current forces.

(a) Currents can exert forces on both sheet-pile and rubble-mound groins; current caused forces, however, are usually small when compared with the forces due to waves. On sheet-pile groins, forces may result from the longshore current's impingement on the groin or from seaward flowing rip currents along the groin itself. Rip currents can cause an additional lateral force (along the axis of the groin) on a groin's lateral support piling.

(b) Current forces also act on rubble-mound groins both as longshore currents flowing over low groins and as seaward flowing rip currents along a groin's flank. Normally the stone weight necessary for stability against currents will be much less than the stone weight necessary for stability against wave action. Appendix IV of EM 1110-2-1601 discusses current forces on rubble and riprap bank protection.

(3) Earth forces. In addition to wave forces, forces due to the buildup of sediment and difference in sand elevation from one side of a sheet-pile groin to the other are important. The resulting earth forces coupled with wave forces establish maximum lateral forces and maximum bending stresses in cantilevered sheet-pile groins. Generally, the maximum sand elevation difference results in the maximum lateral force per unit groin length. The lateral earth force is due to a combination of both active and passive earth pressures acting on the updrift and downdrift sides of a groin. Active earth pressure occurs when there is a rotation or deflection of the pile groin. Active earth pressure acts in the direction of the deflection. Passive earth pressure develops to resist deflection of the groin and acts opposite to the direction of the deflection. The design of cantilevered sheet-pile walls is discussed in most texts on soil mechanics such as Hough (1957) and Terzaghi and Peck (1967). Also see EM 1110-2-2502, which discusses the design of vertical retaining walls. Earth retaining walls experience similar forces.

(4) Ice forces.

(a) Except for the Great Lakes, Alaska, and other freshwater bodies in northern latitudes, ice forces on groins are not important. On the Great Lakes and other freshwater bodies, however, horizontal ice forces on groins can result from a crushing and/or bending ice

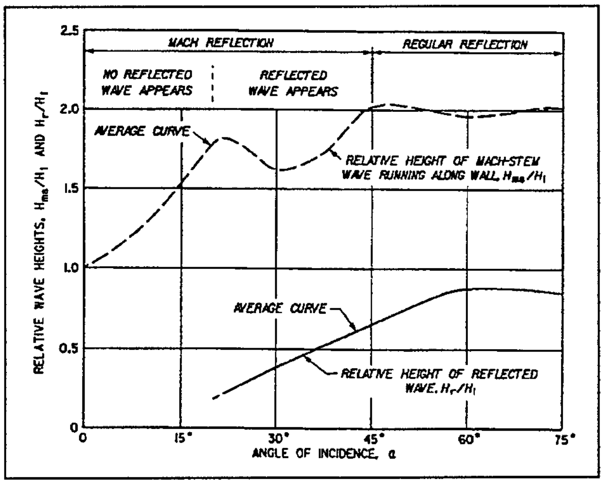


Figure 3-5. Reflection coefficient for Mach-stem reflection of solitary wave (Perroud 1957)

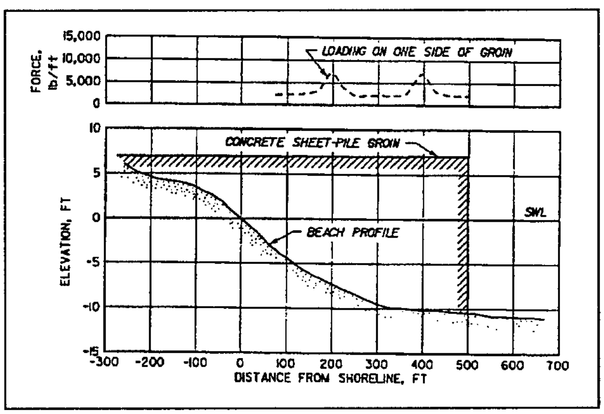


Figure 3-6. Loading diagram, cnoidal waves running along a cantilevered sheet-pile groin (Weggel 1981)

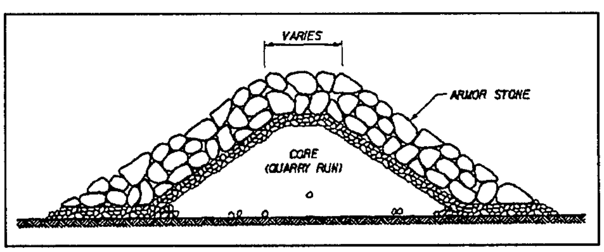


Figure 3-7. Typical cross section of a rubble-mound groin

failure of laterally moving ice sheets, impact by large floating ice masses, and by plucking forces on riprap and rubble. Vertical forces arise because of the weight of ice frozen on structures following lowering of the water level or due to water spray, and buoyant uplift forces due to an increase in water level. Fortunately, maximum ice and wave forces do not usually occur at the same time since ice shelters a structure from wave action.

(b) For groins closely spaced along a shore or closely spaced structural elements on a single groin, the expansion of a large ice sheet due to a temperature increase can lead to forces and deflections. The SPM (1984, Chapter 7, Section VI) and EM 1110-2-1612 provide information on the physical characteristics of ice and potential ice forces.

(5) Other forces.

(a) Other forces a groin might experience include impact forces due to wave-carried debris and small craft collisions. The magnitude of these forces is difficult to predict because the cause of the impact and the mass of the impacting body are not known a priori. If debris is suspected to be a problem, appropriate levels of conservatism should be included in the design.

(b) A groin may have to be designed to withstand forces that might occur only during construction; e.g., the groin may have to carry construction equipment or there may be surcharge due to temporary fill. These forces may be critical and exceed forces due to other more routine causes such as waves and currents.

b. *Structural analysis.*

(1) Fatigue. Wave action on sheet-pile groins located in coastal regions produce unique cyclic loading conditions relative to conventional vertical retaining walls on inland waterway systems. The stress range and number of cycles produced by the waves along with any unique framing conditions should be considered in the structural design of a groin. Fatigue considerations are discussed in the ASCI Steel Construction manual, *Allowable Stress Design* (1989).

(2) Fracture. Steel sheet piles used for groins may have high carbon equivalents and transition temperatures below the ambient project temperature. Consequently, the possibility of brittle fracture and stress corrosion cracking

should be considered in the structural design. Fracture considerations are discussed in Barsom and Rolfe (1987).

3-4. Design Process.

a. Prototype examples.

(1) One of the best predictors of a planned groin's performance is the performance of existing nearby groins or groins in similar wave and longshore transport environments. They can provide both functional and structural performance data. Nearby groins are usually sited in essentially the same wave and longshore transport environment and are acted upon by essentially the same forces. Shoreline response can be expected to be similar, with appropriate adjustments due to differences in exposure and shoreline alignment.

(2) Functional performance can be judged by observing the shoreline updrift and downdrift of an existing groin to determine the postconstruction shoreline that might be expected. Similarly, seasonal changes in shoreline alignment can be assessed. Care should be exercised, however, in extrapolating the observed behavior of a single isolated groin to the behavior of groins in a groin field. For the former, a long updrift beach can provide a source of sand, and the updrift fillet will continue to build, whereas for the latter, the sand supply is limited by the amount of sand within the groin compartment. In this case, the shoreline response will be more rapid. Even though the rate of response will be faster for the groin compartment, the general shoreline alignment should be about the same. If the rates of updrift accumulation and downdrift erosion at an existing groin have been monitored, information on longshore sand transport rates can be obtained, which in turn can be used to predict the rate at which the groins will fill.

(3) If a groin field is under construction, the sequence of construction is important, especially if beach fill is not a part of the project and the groins are expected to fill by natural longshore transport. The downdrift groin should be constructed first and allowed to fill before the next updrift groin is constructed; i.e., construction should proceed from the most downdrift groin in an updrift direction. If the sequence of construction is reversed, the groin farthest downdrift will take a long time to fill because each of the updrift compartments must fill before sand will start to bypass to the next downdrift compartment.

(4) If a groin field is to be constructed, monitoring the filling rate of the first groin during construction can provide timely transport rate data for comparison with transport rates adopted for design. The observed rate might then be used to modify or revise the construction schedule of subsequent groins. The designer is cautioned here, however, about using short-term data without some evidence of its applicability.

(5) Observations of the performance of existing nearby groins of a similar type of construction will indicate if rip currents will form along the proposed groins.

(6) Some indication of a groin's structural performance can also be obtained by noting the condition of nearby groins. Nearby groins, if they terminate offshore in the same depth of water, will be subjected to the same wave environment and thus to approximately the same wave forces. Structural damage to existing groins can be used as an indicator of the wave environment. The armor stone size of a damaged rubble-mound groin can be determined and used to check the armor stone size of a proposed groin design. Similarly, undamaged rubble-mound groins can provide an upper limit on required armor stone size. The frequency at which existing groins sustain damage and their level of performance in a damaged state can help establish an acceptable level of design. (It may not be economical to design for a very large wave because it will rarely occur.)

(7) Information on the structural performance of other types of groins can also be obtained by observing the performance of existing structures. For example, some measure of the potential for structural deterioration, corrosion, abrasion, etc., can be obtained by noting the effects of these processes on nearby structures.

b. Model Investigations.

(1) Physical models.

(a) Physical models may be used for both functional and structural design of groins. Hydraulic model tests, their design, conduct, interpretation, etc., are presented in detail by Hudson et al. (1979). Additional information can be found in EM 1110-2-2904.

(b) For functional design, three-dimensional hydraulic models that include the effects of both waves and tides are generally required. A simple fixed-bed model can establish current patterns due to waves and tides; however, a tracer material (sand simulant) must be introduced into the model to model the effect of a project on the shoreline and on sedimentation patterns. Such fixed-bed models with sediment tracers have been used with

moderate success to qualitatively (and to a very limited extent, quantitatively) describe shoreline evolution and define areas of scour and deposition. Model materials such as coal, walnut shells, and plastic beads have been used as tracers. Scaling laws and the relationship between time in the model and prototype are not precisely known. One difficulty is to accurately reproduce the prototype wave environment in the model; at best the model wave environment is simulated by a few different wave conditions that are assumed to characterize the prototype environment.

(c) Moveable bed models can be used to study groin behavior; however, in most cases, generalized research models rather than site-specific models have been used. Fine sand is often used as the moveable bed material; however, other materials have also been used. Often a distorted model, where the vertical scale is exaggerated, must be used. This usage further complicates the scaling laws needed to compare the model with the prototype. Moveable bed models are expensive to build and operate, and model results can be difficult to translate into prototype performance. In view of the limited quality of the information they provide, they are sometimes difficult to justify for projects involving groins.

(d) Physical models to study the structural performance of a groin's design may be justified where many similar rubble-mound groins are to be built or where the wave environment is particularly severe. Structural tests of groin types other than rubble-mound groins are not common. The wave and earth loading on sheet-pile groins is easier to define than the loading on and stability of rubble-mound groins. If stability testing is indicated, three-dimensional tests or tests in an L-shaped flume are necessary because waves usually approach a groin nearly head-on. It is usually the groin's head that must absorb the brunt of the wave attack, and it is the most critical element to be modeled. A model of the groin is built in an L-shaped or wide laboratory flume or basin and subjected to increasingly higher waves until armor units start being displaced by waves. The wave height that just initiates armor layer damage is established as the zero-damage wave height. A stability coefficient and a scaling relationship such as Hudson's equation are then used to determine the corresponding prototype armor unit size.

(2) Numerical models.

(a) Numerical computer models that model the effect of coastal structures on shorelines have evolved to the point where they can be used to predict the effect of groins and other coastal structures on a shoreline. One-line numerical models predict the location of a single contour line, usually the shoreline (LeMéhauté and Soldate 1980, Perlin and Dean 1979, Kraus 1983, Hanson and Kraus 1989). These models are the sediment budget equations applied to a finite difference representation of a stretch of shoreline. The equations express the conservation of sand with an equation of motion that relates sediment transport rates to incident wave conditions. For multiple-line models, onshore-offshore or crosshore transport is also considered. Onshore-offshore transport is related to wave conditions and to the local bottom slope. The incident wave conditions determine an equilibrium beach profile, and the existing profile moves toward that equilibrium. Wave conditions usually change before the equilibrium profile is reached so that the profile is continually adjusting toward a new equilibrium.

(b) Numerical models have the advantage of simulating shoreline response to time-varying wave conditions. The time-history of the shoreline, including its seasonal variations, can be computed if a time-history of the wave environment is available or can be synthesized. Wave data such as WIS hindcasts (Jensen 1983) can be used as input for such models. Numerical models also offer the potential of studying shoreline response to waves and water levels due to major storms (Larson et al. 1990).

(c) A one-line numerical model like GENESIS (Hanson and Kraus 1989) can be used to predict shoreline evolution following the construction of shore stabilization structures such as groins, offshore breakwaters, and seawalls. A description of GENESIS is provided in Appendix D.

c. Empirical relationships.

(1) There are few empirical relationships governing the design of groins and groin fields. For example, one simple empirical relationship is the recommendation that groin spacing be two to three times the groin length measured from the bermline to the seaward end of the groin.

(2) Another empirical rule deals with estimating the amount of sand bypassing a groin. For long, high groins that extend seaward to a depth of -3 meters or more below MLW or MLLW, all longshore transport is trapped. For high groins extending to depths of from -1.2 to -3.0 meters, about 75 percent of the longshore transport is trapped. Also, for low groins extending to less than -3.0 meters, 75 percent of the longshore transport is trapped. For high, short groins extending seaward to

depths of only -1.2 meters, 50 percent of the longshore transport is trapped. Note, however, that as a groin system fills, the water depth at the groin's seaward end changes so that the amount of sand bypassing the structure is a function of both time and incident wave conditions.

Chapter 4
Nearshore Breakwaters

4-1. Purpose

a. Scope. The purpose of this chapter is to provide design guidance on nearshore breakwaters and submerged sills. Their advantages and disadvantages are presented along with how they influence waves, what their effect is on the shoreline, and when they are viable options as effective shoreline stabilization methods. General information and a review of detached breakwater projects in the United States can be found in Dally and Pope (1986), Pope (1989), and Kraft and Herbich (1989).

b. Nearshore breakwaters. Offshore breakwaters are generally shore-parallel structures that effectively reduce the amount of wave energy reaching the protected stretch of shoreline. They can be built close to the shoreline they are intended to protect, in which case they are called nearshore breakwaters, or they can be built farther from shore. When used for beach stabilization, breakwaters function to reduce wave energy in their lee and thus reduce the sediment carrying capacity of the waves there. They can be designed to prevent the erosion of an existing beach or a beach fill, or to encourage natural sediment accumulation to form a new beach. Figure 4-1 depicts the basic characteristics of a single detached breakwater.

c. Submerged sills. Submerged sills are also generally shore-parallel and built nearshore. Their purpose is to retard offshore sand movement by introducing a structural barrier. The shore-parallel sill interrupts normal offshore sediment movement caused by storm waves; however, it may also interrupt the onshore movement caused by "beach building" waves. The sill introduces a discontinuity into the beach profile so that the beach behind it is at a higher elevation (and thus wider) than adjacent beaches. The beach is thus "perched" above the surrounding beaches. Figure 4-2 depicts the basic perched beach concept.

d. Difference. A distinction between nearshore breakwaters and submerged sills can be made by noting their effects on waves and sediment transport. Breakwaters act to reduce waves; submerged sills act as barriers to shore-normal sediment motion. The primary characteristics that determine how a structure is classified is the structure's crest elevation. Breakwaters have crest elevations high enough to significantly reduce the

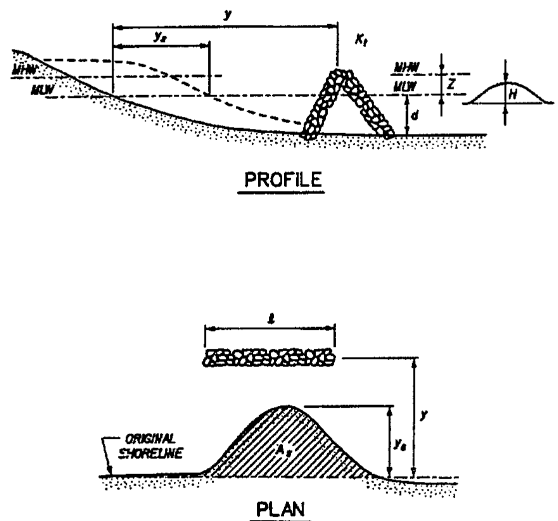


Figure 4-1. Schematic of a single detached breakwater

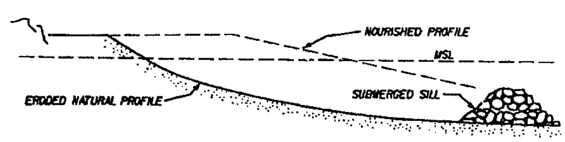


Figure 4-2. Submerged sill and perched beach concept

height of waves transmitted over them. (Waves in the lee of a breakwater can also result from diffraction around the breakwater's end and transmission through the breakwater.) The effect of submerged sills on waves is relatively small because their crest elevation is at or below the water level. Their crest may be exposed at low tide; however, at most stages of tide, they are submerged. While sills may trip some large waves into breaking, they simply provide a barrier to onshore and offshore sediment movement at one point on the beach profile.

4-2. Design Objectives

The primary design objective of a nearshore breakwater or submerged sill system is to increase the longevity of a beach fill, provide a wide beach for recreation, and/or afford protection to upland areas from waves and flooding. In addition, adverse effects, usually erosion along downdrift beaches due to a breakwater's halting or reducing the normal longshore transport, should be minimized.

a. Advantages of breakwaters. Nearshore breakwaters offer several advantages over other beach stabilization structures. First, if properly designed, they effectively control erosion and retain sand on a beach. Second, they reduce the opportunity for rip currents to form and thus reduce offshore sediment losses. Third, they reduce the steepness of waves in their lee and encourage landward sand transport. Fourth, they reduce wave heights along a beach.

b. Disadvantages of breakwaters. Breakwaters also have several disadvantages. There is only a limited amount of US prototype experience with nearshore breakwaters for shoreline protection, although Japan and several Mediterranean countries have had extensive experience with these structures. In addition, design guidance, especially in the planning stages of a project, is somewhat limited. Because they are located offshore, nearshore breakwaters can be expensive to build and may require the use of temporary trestles or barge-mounted construction equipment. Similarly, they may be expensive to maintain because of their offshore location. The gaps between a series of breakwaters can channel flow and sediment offshore if water levels behind the breakwaters build up as a result of wave overtopping. Relatively high offshore velocities through these gaps can scour the bottom unless riprap armoring is provided. Breakwaters can also be a total barrier to longshore sand transport unless care is taken to ensure that some wave energy is available behind them to transport sand. Thus, they can totally halt the flow of sand to downdrift beaches and cause erosion there. Breakwaters can also be hazardous to bathers and swimmers if they climb on the structures or get caught in offshore flows. They can also reduce the potential for recreational surfing in the project area.

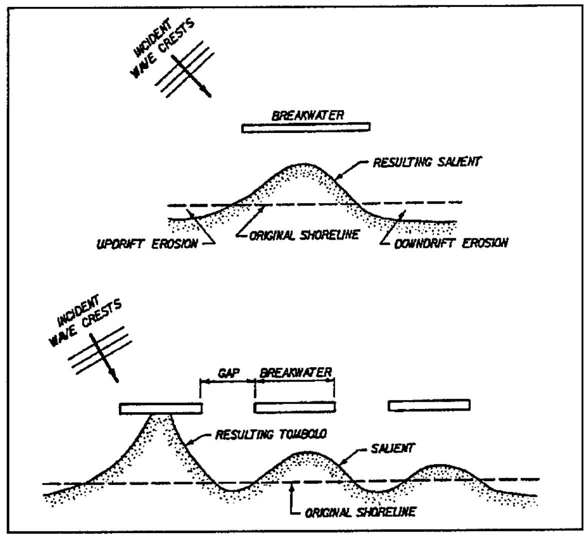


Figure 4-3. types of shoreline changes associated with single and multiple breakwaters and definition of terminology

c. Beach planform. A primary consideration in the design of a nearshore breakwater for beach stabilization purposes is the desired planform and beach width behind the breakwater. Basically, three different types of shorelines can develop behind a breakwater or a system of breakwaters (Figure 4-3). If the breakwater is close to shore, long with respect to the length of the incident waves, and/or sufficiently intransmissible to the average waves, sand will continue to accumulate behind the breakwater until a tombolo forms; that is, the shoreline continues to build seaward until it connects with the breakwater. If a tombolo forms, longshore transport is stopped until the entire updrift beach fills seaward to the breakwater and sand can move around its seaward side. The breakwater-tombolo combination functions much like a T-groin. If the breakwater is far from shore, short with respect to the length of the incident waves, and/or relatively transmissible, the shoreline will build seaward, but is prevented by wave action and longshore currents from connecting with the breakwater. The shoreline bulge that forms is termed a "salient." If a salient forms, longshore sand transport rates are reduced; however, transport is not completely stopped. The third beach type is termed limited shoreline response in which little beach planform sinuosity is experienced, possibly due to a lack of adequate sediment supply. The final shoreline configuration and its location depend on the geometry of the breakwater system, the wave environment, the longshore transport environment, and the amount of available sand. The variability of wave height, period, and direction coupled with the geometry of the breakwater system are all important in determining the final equilibrium planform of the beach.

d. Types of nearshore breakwaters. Nearshore breakwaters can be classified by type of construction and by their planform geometry and crest elevation. There are four basic forms of nearshore breakwaters for shore stabilization. They are a single detached breakwater, a multiple detached breakwater system, artificial headlands, and a submerged sill structure intended to form a perched beach.

(1) A single detached breakwater generally has a limited range of influence and thus protects only a local

reach of shoreline. However, a significant distance of shoreline updrift and downdrift of the breakwater can be affected if a tombolo forms at the structure. Critical design dimensions for a single breakwater are its length, distance offshore, and crest elevation. These dimensions determine whether a tombolo will form and whether the longshore transport rates will prevail following construction.

(2) A multiple breakwater system can be constructed to protect a longer stretch of shoreline. If properly designed, a multiple breakwater system can continue to maintain a reduced rate of longshore transport past a project area, thus minimizing downdrift erosion. Critical design dimensions for multiple breakwater systems are the length of the individual breakwater elements, distance offshore, distance between breakwaters (gap width), and crest elevations. The shape of the resulting shoreline and the amount of transport through the project area depend on these parameters.

(3) Offshore breakwaters have been used as artificial headlands in an attempt to create stable beaches landward of the gaps between the structures (Silvester 1970, 1976; Chew et al. 1974; USAED, Buffalo 1986; Pope 1989; Hardaway and Gunn 1991). A definition sketch for an artificial headland breakwater system is provided in Figure 4-4. As opposed to detached breakwaters where tombolo formation is often discouraged, artificial headland systems are designed to form a tombolo. Artificial headland design parameters include the approach direction of dominant wave energy, length of individual headlands, spacing and location, crest elevation and width of the headlands, and artificial nourishment (Bishop 1982; USAED, Buffalo 1986).

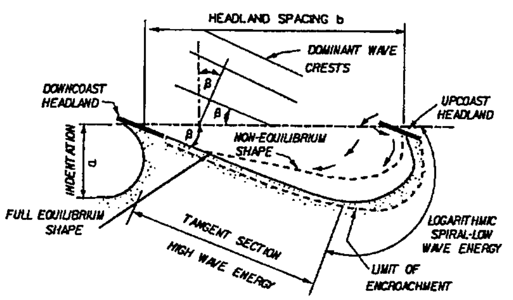


Figure 4-4. Definition sketch of an artificial headland breakwater system and beach planform

(4) Submerged sills can be classified as nearshore breakwaters with crest elevations that are below the mean tide level. They can be built with or without shore-return structures to connect the offshore sill with the shoreline. The shore-return structures and sill hold a beach fill within a boxlike compartment, with the shore returns functioning like groins. Little to no documented experience exists for submerged sills and perched beaches along the exposed ocean coastlines of the United States. There has been some limited experience with perched beach sills in sheltered waters (Dunham, et al., 1982; Douglass and Weggel 1987). This experience suggests that submerged sills slow offshore losses from an area, but that periodic nourishment of the compartment is still necessary to maintain a wide beach. Important design parameters include the sill length, distance offshore, crest elevation, and whether or not to include shore-return structures in the design.

e. Structural effects and design parameters.

(1) Length of shoreline to be protected.

(a) The length of shoreline protected by a single breakwater (and also the downdrift length of shoreline that might be adversely affected by a single breakwater) depends on whether or not the breakwater forms a tombolo. If a tombolo forms in a continuous littoral system, the effect of the breakwater will be to accumulate sand along updrift beaches and to starve downdrift beaches. If located in an area where the net longshore transport is close to zero, the breakwater's range of influence will be limited to the general vicinity of the structure, and the effects may not extend very far updrift or downdrift. If a longer portion of the shoreline must be protected, a system of several breakwaters spaced along the shoreline with gaps between them must be constructed. Building a single long breakwater will not achieve the same result, but will result in the formation of a single tombolo or of two tombolos, one extending seaward from shore to each end of the breakwater. The resulting lagoon enclosed by the breakwater and tombolos is usually undesirable. A multiple breakwater system with gaps also reduces the amount of material needed for construction. In most cases, when there is a net direction of longshore transport, tombolos are unwanted because of the downdrift erosion caused by totally interrupting longshore transport. Generally, a system of multiple nearshore breakwaters is needed to protect a long reach of shoreline while still maintaining some longshore transport to minimize erosion along downdrift beaches.

(b) Wave heights behind a nearshore breakwater can be significantly reduced. Waves in the lee of a

breakwater get there by transmission through the structure if it is permeable, regeneration in the lee of the structure by overtopping waves, and diffraction around the ends of the breakwater. If the breakwater's crest elevation is high and it is impermeable, diffraction is the primary source of wave energy in the shadow zone. Wave diffraction is discussed in the SPM (1984, Chapter 3, Section IV). For a detached breakwater, waves propagate around each end of the breakwater and interact in its lee. Wave heights become smaller farther behind the breakwater. If the incident waves are nearly monochromatic, they interact constructively or destructively behind the breakwater, depending on whether the crest and trough of the waves coming around each end are in or out of phase with each other. Thus, there are regions behind the breakwater where monochromatic waves nearly cancel each other and other areas where they reinforce each other. If a range of wave periods is present, as it often is in the prevailing wave spectrum, a more uniform distribution of wave heights prevails in the breakwater's lee. As the direction of incoming waves changes, the salient in the sheltered area behind the breakwater responds by repositioning itself in the region to the structure's lee. A diffraction analysis should be used to determine the approximate shoreline configuration behind a breakwater. Studies indicate that if the isolines of the $K' = 0.3$ diffraction coefficients are constructed from each end of the breakwater for a range of incident wave directions and they intersect seaward of the postproject shoreline, a tombolo will not form (Figure 4-5) (see Walker et al. 1980). More simply, this is ensured if the breakwater lies more than one half the breakwater's length seaward of the postproject shoreline, i.e. after placement of beach fill if that is part of the project. Waves coming around each end of the breakwater meet each other before the undiffracted incident wave (outside of the breakwater's shadow) reaches the shoreline. The postproject shoreline can be estimated by drawing the pattern of the diffracted wave crests behind the breakwater and smoothing the crest pattern to balance the amount of sediment available.

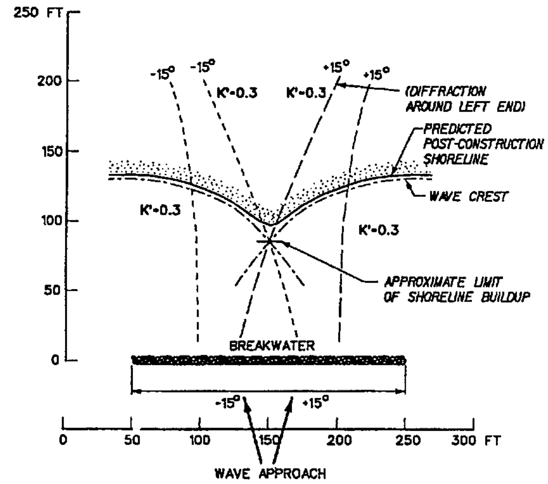


Figure 4-5. Estimate of post project shoreline behind a detached nearshore breakwater, isolines of diffraction coefficient, $K' = 0.3$

(2) Types of construction.

(a) Most US and foreign nearshore breakwaters built for shore protection have been rubble-mound structures. Several structures have been built of steel sheet-pile cells in the Great Lakes; however, these structures were not intended to function as shore protection, but rather to protect a harbor entrance from waves (for example, Vermilion Harbor, Ohio). Their effect on adjacent shorelines, however, has been similar to that of shore stabilization breakwaters. Rubble-mound construction of nearshore breakwaters is advantageous since rubble-mound structures dissipate more incident wave energy and are relatively easy to construct in the nearshore zone. Several patented shore protection devices that function like nearshore breakwaters have been built, mostly in sheltered waters. Some of these have been tested under the Shoreline Erosion Control Demonstration Act (Dunham et al. 1982). Several have been built of precast reinforced concrete units; others have been built of concrete blocks and sand-filled geotextile tubes and bags. Refer to EM 1110-2-2904 for further guidance on the design of rubble-mound and other type structures.

(b) Submerged sills of various types have been built in sheltered waters. Sand-filled bags, timber sheet piles, and sand-filled precast concrete boxes have been used for sill construction. There does not appear to be a discernible difference in functional performance between the various types of sills; however, a sand-tight rubble-mound sill is recommended for perched beaches because of its ability to dissipate wave energy.

(3) Crest elevation.

(a) Crest elevation determines the amount of wave energy transmitted over the top of a nearshore breakwater or submerged sill. High crest elevations preclude overtopping by all but the highest waves

whereas low crest elevations allow frequent overtopping. Generally, low crests allow more wave energy to penetrate into the lee of the breakwater. Occasional overtopping of a nearshore breakwater by storm waves can prevent tombolo formation or remove a tombolo once it has formed. For an artificial headland system, the amount of overtopping should be minimized to encourage tombolo formation. Wave transmission by overtopping is discussed in the SPM (1984, Chapter 7, Section II). Prediction of irregular wave overtopping of structures is discussed in Ahrens (1977).

(b) For a submerged sill, crest elevation determines the elevation and spatial extent of the perched beach that can be maintained behind the sill. Higher sills also have more effect on incident waves. While the primary purpose of a submerged sill is to retain sand, it also triggers breaking by some waves and reduces wave energy levels on the perched beach. As the sill elevation is increased, it begins to function more like an offshore breakwater; that is, its effect on waves increases.

(4) Circulation and modification of currents.

(a) Construction of offshore breakwaters and sills will result in significant changes in the nearshore current system. On a natural beach, shore-parallel longshore currents are generated by waves approaching the shoreline at an angle. If breakwaters are built, the driving force for the currents is intercepted by the breakwater along part of the shoreline. The prevailing longshore current, unless maintained by its inertia, will slow or stop when it moves into the sheltered area behind the breakwaters. The sand carrying capacity of the current and the wave agitation that suspends sediment so it can be carried by the current are reduced. A breakwater's length and distance from shore are critical in determining its effect on longshore currents and sediment transport. A long breakwater will cause the longshore current to slow and spread laterally and will shelter a long reach of shoreline from wave agitation.

(b) If the breakwater crest elevation is low enough to allow overtopping, water carried over the breakwater will raise the water level behind it and cause flow around the breakwater. In multiple breakwater systems, overtopping causes a net seaward flow of water through the gaps. Seelig and Walton (1980) present a method for estimating the strength of the seaward flowing currents. Return currents can be reduced by raising the breakwater crest elevation, enlarging the gaps between segments, or increasing structure permeability. For permeable breakwaters, some flow is also carried seaward through the breakwater itself.

(5) Effect on wave environment.

(a) Breakwaters reduce the amount of wave energy reaching the shoreline. Wave heights in the lee of a breakwater are much lower than they are in the exposed area seaward of the breakwater. Waves in the lee of a breakwater are determined by three processes: diffraction around the breakwater ends, wave transmission by overtopping, and wave transmission through the structure. Local diffracted wave heights are determined primarily by their exposure and distance from the breakwater's ends or, in the case of a multiple breakwater system, by their location relative to the breakwater gaps (see SPM (1984), Chapter 2, Section IV). Wave heights due to overtopping are determined by the breakwater crest elevation. Wave transmission through a breakwater is determined by its permeability (SPM (1984) Chapter 7, Section II; Madsen and White 1976; Seelig 1979, 1980). The Automated Coastal Engineering System (Leenknecht et al. 1990) provides an application to determine wave transmission coefficients and transmitted wave heights for permeable breakwaters with crest elevations at or above the still-water level. This application can be used with breakwaters armored with stone or artificial armor units.

(b) Wave conditions seaward of a breakwater are determined by its reflection characteristics. Reflected waves interact with incident waves to cause a partial standing wave pattern seaward of a breakwater. Agitation of bottom sediments by standing waves can cause scour and undermining seaward of the breakwater and contribute to other foundation problems. Reflection characteristics are in turn determined by breakwater permeability, crest elevation, and type of construction. Permeable, low-crested, rubble-mound breakwaters are the least reflective structures; however, they can allow significant amounts of energy to propagate through them. Rubble-mound structures dissipate wave energy by inducing fluid turbulence in their interstices.

(6) Effect on longshore transport.

(a) Nearshore breakwaters reduce longshore transport rates by sheltering a reach of shoreline from waves. Much like a groin, the breakwater forms a partial or total barrier to longshore transport. The reduction in transport capacity is determined by both a reduction in wave height in the breakwater's lee and by redirection of wave crests by diffraction around the breakwater's

ends. Long single breakwaters or closely spaced multiple breakwaters can form a near complete barrier to longshore transport. If a tombolo forms, transport is almost totally interrupted, with the exception of transport seaward of the breakwater. For breakwaters where only salients develop, longshore transport rates can be adjusted to meet desired design objectives. Sediment budget analyses should be made to determine the effect of a transport rate reduction on both updrift and downdrift beaches under postproject conditions. Adjusting the length, distance offshore, and crest elevation of a single breakwater will vary the resulting longshore transport rate. For multiple breakwater systems, gap width may also be modified. A fixed-bed physical model with a sediment simulant tracer can be useful in estimating and comparing pre- and postproject transport rates for various cases.

(b) In general, the effect of submerged sills on longshore sediment transport is relatively small. Since there is a small reduction in incident wave energy, there will be some reduction in transport rates within a perched beach. In cases where the breaking wave angle is relatively small, there may be a more significant effect on longshore transport. If shore-return structures are included in a perched beach design, they will affect longshore transport similar to low groins, and the rate of longshore transport into and out of the perched beach area will be reduced.

(7) Effect on onshore-offshore transport.

(a) Nearshore breakwaters can reduce offshore sand transport. Wave heights in a breakwater's lee are reduced, and their direction is changed. Lower wave heights result in waves with a lower wave steepness (wave-height-to-wavelength ratio) and are therefore more likely to transport sand onshore than offshore. For multiple breakwater systems, offshore sand losses are reduced; however, overtopping can result in a net seaward flow of water and sand through the gaps between breakwater segments. These currents usually occur when the structure is nearly impermeable and low crested so that the water transmitted by overtopping can return only through the gaps or around the ends of the structure. The breakwater can also reduce onshore sediment movement. Following breakwater construction, a new equilibrium between onshore and offshore transport will be established.

(b) Submerged sills are intended to reduce the rate of offshore sand transport. They establish a location on the beach profile across which both offshore and onshore transport is much reduced from what it would be across a normal profile. While the sill is intended to reduce offshore losses during storm wave conditions, it also reduces onshore movement during beach-building, low-steepness wave conditions. The sill's net effect on onshore-offshore transport processes has not been quantitatively established; consequently, it is not known whether the sill's overall effect is beneficial or detrimental. A laboratory study by Sorensen and Beil (1988) investigated the response of a perched beach profile to storm wave attack.

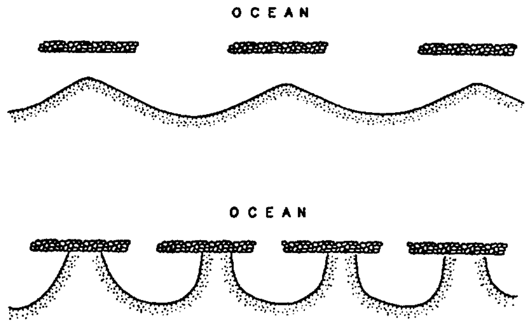
f. Design to meet functional objectives.

(1) A single detached breakwater or multiple breakwater system generally has as its primary objectives to increase the life of a beach fill, provide a wide beach for recreation, and/or protect upland development. To establish and protect a relatively short reach of shoreline (on the order of several hundreds of feet), a single breakwater can provide the needed sheltering. If a tombolo is allowed to form acting as a littoral barrier, a single breakwater's effects can extend a great distance upcoast and downcoast.

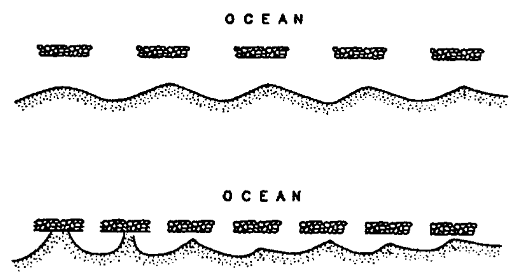
(2) If located in an area where the net transport is almost zero, but where the gross transport is not zero, the breakwater's major effects will be limited to the general vicinity of the breakwater itself. Minor effects, however, can extend to significant distances. The time period for the effects of a breakwater to be observed along updrift and downdrift beaches depends on both the net and gross transport rates. For large transport rates, the effects are felt quickly; for low rates, the effects may take years to appear.

(3) If a significant length of shoreline must be protected, a multiple breakwater system should be considered. The number of breakwaters, their size, and the gap width between them depend on the wave environment and the desired shape of the shoreline behind them. A few long, widely spaced breakwaters will result in a sinuous shoreline with large amplitude salients and a spatial periodicity equal to the spacing of the breakwaters; that is, there will be a large salient behind each breakwater (Figure 4-6a). Numerous, more closely spaced breakwaters will also result in a sinuous shoreline, but with more closely spaced, smaller salients (Figure 4-6b).

(4) Wide gaps permit more wave energy to penetrate into the area behind the breakwaters, thus maintaining



a. With a few relatively long, widely spaced segments



b. With more numerous, shorter, closely spaced segments

Figure 4-6. Multiple breakwater system

some level of longshore sand transport. The ratio of gap width to the sum of breakwater length and gap width for various prototype projects (the fraction of the shoreline directly open to waves through the gaps, termed the "exposure ratio") ranges from about 0.25 to 0.66. Table 4-1 provides examples of various prototype projects and their associated exposure ratios. Projects like Presque Isle, PA, and East Harbor State Park, OH, where the purpose is to contain a beach fill within fixed project boundaries have larger exposure ratios. Comparatively, the exposure ratio at Lakeview Park, Lorain, OH, is 0.36, and at Winthrop Beach, MA, where the gaps were included to allow for small craft navigation, the ratio is 0.25.

(5) The postproject shoreline configuration can be determined from diffraction analyses using a range of wave conditions characteristic of the site. The design process is one of trial and error. A trial breakwater configuration is assumed based on past experience at existing breakwater systems. Then the trial configuration is evaluated to determine if it will satisfy the project's objectives. Its effect on the shoreline and on the overall sediment budget of the project area and adjacent beaches is evaluated. The trial configuration is adjusted and the modified project's effects evaluated. Evaluation tools for proposed breakwater configurations include the interpretation of diffraction analyses, overtopping analyses, and other manual computations; physical model tests of the proposed project configuration; and numerical computer simulations of shoreline evolution. Because of the limited experience with prototype detached breakwaters in the United States, a great deal of engineering judgment and comparison with the few existing breakwater projects is necessary.

(6) Dimensional analysis can provide some insight into the design of single and multiple nearshore breakwater systems. A more detailed section on dimensional analysis of detached breakwaters and an example application can be found in Appendix D.

g. Empirical relationships for breakwater design.

(1) Summary of relationships.

(a) The functional design and prediction of beach response to single and segmented detached breakwaters systems have been the subject of numerous papers and reports (SPM 1984; Gourlay 1981; Ahrens and Cox 1990; Dally and Pope 1986; Suh and Dalrymple 1987; Nir 1982; Noble 1978; Inman and Frautschy 1966). A number of these references have been reviewed in Rosati (1990) and are summarized in Table 4-2. A design procedure developed by the Japanese Ministry of Construction (JMC) (1986) has been summarized by Rosati and Truitt (1990). Most references present morphological information on when tombolos will form and when minimal beach response to breakwater construction can be expected. These conditions are usually specified in terms of the dimensionless breakwater length, ℓ/y, where y is the distance from the average shoreline; or the breakwater length-to-wavelength ratio, ℓ/gT^2, where g is the acceleration of gravity and T is the wave period. The other dimensionless parameters given in Appendix D are also important and play a role in determining how the shoreline responds to nearshore breakwater construction.

(b) Conditions for tombolo formation cited by various investigators are given in Table 4-3. The conditions for salient development are given in Table 4-4, and the

Table 4-1
"Exposure Ratios" for Various Prototype Multiple Breakwater Projects*

Project	Exposure Ratio
Winthrop Beach, MA	0.25
Lakeview Park, Lorain, OH	0.36
Castlewood Park, Colonial Beach, VA	0.31 to 0.38
Central Beach, Colonial Beach, VA	0.39 to 0.45
East Harbor State Park, OH	0.56
Presque Isle, Erie, PA	
(experimental prototype)	0.56 to 0.66
(hydraulic model)	0.60

* The "exposure ratio" is defined as the ratio of gap width to the sum of the breakwater length and gap width. It is the fraction of shoreline directly exposed to waves and is equal to the fraction of incident wave energy reaching the shoreline through the gaps. A "sheltering ratio" that is the fraction of incident wave energy intercepted by the breakwaters and kept from the shoreline can also be defined. It is equal to 1 minus the "exposure ratio."

conditions for limited shoreline response are given in Table 4-5.

(c) Other empirical relationships for other variables have been proposed. For example, Suh and Dalrymple (1987) propose the following relationships for the length of the salient behind a single breakwater:

$$\frac{y_s}{\ell} = 0.156 \text{ for } \frac{y_b}{y} < 0.5 \qquad (4\text{-}1)$$

$$\frac{y_s}{\ell} = 0.317 \text{ for } 0.5 < \frac{y_b}{y} < 1.0 \qquad (4\text{-}2)$$

$$\frac{y_s}{\ell} = 0.377 \text{ for } \frac{y_b}{y} > 1.0 \qquad (4\text{-}3)$$

where y_b is the distance from shore to the breaker line and y_s is the distance to the salient end from the average shoreline. Behind multiple breakwaters, Suh and Dalrymple (1987) propose,

$$y_s = 14.8\, y\, \frac{by}{\ell^2} \exp\left[-2.83 \sqrt{(by/\ell^2)}\right] \qquad (4\text{-}4)$$

for the length of the salient, where b is the gap width.

(2) JMC method.

(a) Rosati and Truitt (1990) have summarized a procedure developed by the JMC for the design of a system of nearshore breakwaters. The procedure, developed from observations of the performance of a number of Japanese prototype breakwaters, results in a system of relatively short breakwaters located close to shore. Beach nourishment was not included in most of the prototype projects on which the procedure is based. Five different shoreline types were investigated. Type A is for shallow offshore areas, small wave heights, beach slopes of about 1:30, and fine sand. Type B is for beaches with well-developed offshore bars, gentle slopes (1:30), moderate wave heights, and mostly shore-normal incident waves. Type C is for relatively steep slopes (1:15), no offshore bar, moderate wave heights, and beaches of coarse sand and pebbles. Type D is for more steeply sloping beaches (1:3 to 1:10), moderate wave heights, and pebbles. Type E is similar to Type C but with an offshore bar. Detailed descriptions of the coasts for which the procedures were developed are given in Rosati and Truitt (1990). Sufficient data were available to develop detailed design procedures for Type B and C coastlines.

(b) The design wave used in the procedure is the average deepwater height of the five highest "nonstorm" waves occurring in a year, H_{o5}, and the wave period associated with that wave height, T_5. There is currently no simple way to relate this H_{o5} wave to other characteristic waves at a site such as the mean wave height or some other wave from the wave height distribution with a specified return period. H_{o5} is certainly less than the 1-year wave height (the wave height equaled or exceeded at least once in each year) but higher than the average daily wave height.

(c) After selecting the length of the shoreline reach to be protected and the desired shoreline advancement (salient length, y_s), the breaking water depth, d_b, of the H_{o5} wave is calculated using Figure 4-7 with the deepwater values of H_{o5} and L_{o5} (the deepwater wavelength associated with T_5). Calculate the ratio d'/d_b where d' is the water depth at the offshore breakwater estimated using $d' = (d_b + y \tan \beta)/2$ where $\tan \beta$ is the bottom slope. With the ratio d'/d_b, the salient area ratio (SAR) can be found from Figure 4-8. The SAR is given by,

Table 4-2
Summary of Empirical Relationships for Breakwater Design

Reference	Comment
Inman and Frautschy (1966)	Predicts accretion condition; based on beach response at Venice in Santa Monica, CA
Toyoshima (1972, 1974)	Recommends design guidance based on prototype performance of 86 breakwater systems along the Japanese coast
Noble (1978)	Predicts coastal impact of structures in terms of offshore distance and length; based on California prototype breakwaters
Walker, Clark, and Pope (1980)	Discusses method used to design the Lakeview Park, Lorain, OH, segmented system for salient formation; develops the Diffraction Energy Method based on diffraction coefficient isolines for representative waves from predominant directions
Gourlay (1981)	Predicts beach response; based on physical model and field observations
Nir (1982)	Predicts accretion condition; based on performance of 12 Israeli breakwaters
Rosen and Vadja (1982)	Graphically presents relationships to predict equilibrium salient and tombolo size; based on physical model/prototype data
Hallermeier (1983)	Develops relationships for depth limit of sediment transport and prevention of tombolo formation; based on field/laboratory data
Noda (1984)	Evaluates physical parameters controlling development of tombolos/salients, especially due to on-offshore transport; based on laboratory experiments
SPM (1984)	Presents limits of tombolo formation from structure length and distance offshore; based on the pattern of diffracting wave crests in the lee of a breakwater
Dally and Pope (1986)	Recommends limits of structure-distance ratio based on type of shoreline advance desired and length of beach to be protected
Harris and Herbich (1986)	Presents relationship for average quantity of sand deposited in lee and gap areas; based on laboratory tests
JMC (1986); also Rosati and Truitt (1990)	Develops step-by-step iterative procedure, providing specific guidelines towards final breakwater design; tends to result in tombolo formation
Pope and Dean (1986)	Presents bounds of observed beach response based on prototype performance; beach response given as a function of segment length-to-gap ratio and effective distance offshore-to-depth at structure ratio; provides beach response index classification scheme
Seiji, Uda, and Tanaka (1987)	Predicts gap erosion; based on performance of 1,500 Japanese breakwaters
Sonu and Warwar (1987)	Presents relationship for tombolo growth at the Santa Monica, CA, breakwater
Suh and Dalrymple (1987)	Gives relationship for salient length given structure length and surf zone location for single breakwater; based on laboratory tests
Berenguer and Enriquez (1988)	Presents various relationships for pocket beaches including gap erosion and maximum stable surface area (i.e., beach fill); based on projects along the Spanish coast
Ahrens and Cox (1990)	Uses Pope and Dean (1986) to develop a relationship for expected morphological response as function of segment-to-gap ratio
Ahrens (unpublished)	Extends results of Berenguer and Enriquez (1988)

Table 4-3
Conditions for the Formation of Tombolos

Condition	Comments	Reference
$\ell/y > 2.0$		SPM (1984)
$\ell/y > 2.0$	Double tombolo	Gourlay (1981)
$\ell/y > 0.67$ to 1.0	Tombolo (shallow water)	Gourlay (1981)
$\ell/y > 2.5$	Periodic tombolo	Ahrens and Cox (1990)
$\ell/y > 1.5$ to 2.0	Tombolo	Dally and Pope (1986)
$\ell/y > 1.5$	Tombolo (multiple breakwaters)	Dally and Pope (1986)
$\ell/y > 1.0$	Tombolo (single breakwater)	Suh and Dalrymple (1987)
$\ell/y > 2\ b/\ell$	Tombolo (multiple breakwaters)	Suh and Dalrymple (1987)

Table 4-4
Conditions for the Formation of Salients

Condition	Comments	Reference
$\ell/y < 1.0$	No tombolo	SPM (1984)
$\ell/y < 0.4$ to 0.5	Salient	Gourlay (1981)
$\ell/y = 0.5$ to 0.67	Salient	Dally and Pope (1986)
$\ell/y < 1.0$	No tombolo (single breakwater)	Suh and Dalrymple (1987)
$\ell/y < 2\ b/\ell$	No tombolo (multiple breakwaters)	Suh and Dalrymple (1987)
$\ell/y < 1.5$	Well-developed salient	Ahrens and Cox (1990)
$\ell/y < 0.8$ to 1.5	Subdued salient	Ahrens and Cox (1990)

Table 4-5
Conditions for Minimal Shoreline Response

Condition	Comments	Reference
$\ell/y \leq 0.17$ to 0.33	No response	Inman and Frautschy (1978)
$\ell/y \leq 0.27$	No sinuosity	Ahrens and Cox (1990)
$\ell/y \leq 0.5$	No deposition	Nir (1982)
$\ell/y \leq 0.125$	Uniform protection	Dally and Pope (1986)
$\ell/y \leq 0.17$	Minimal impact	Noble (1978)

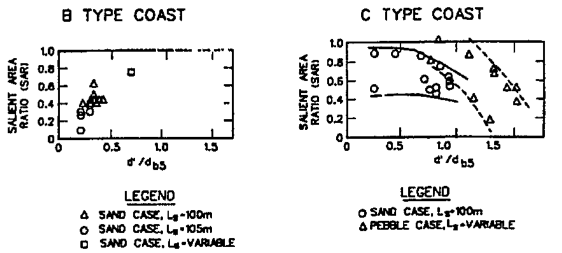


Figure 4-7. Deepwater wave steepness as a function of nearshore steepness for various beach slopes (Goda 1970)

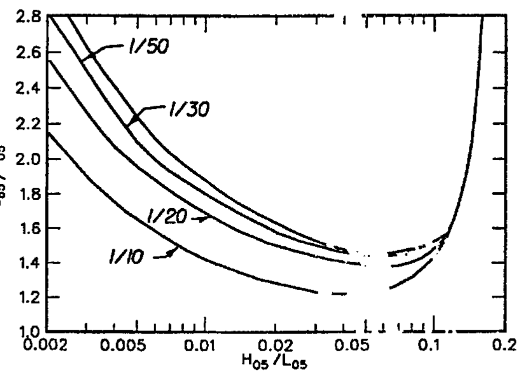


Figure 4-8. Salient area ration as a function of relative water depth, Type B shoreline and Type C shoreline

$$SAR = \frac{\frac{1}{2}(\ell_c y_s)}{y\,\ell} \tag{4-5}$$

in which ℓ_c is the salient length in the longshore direction measured at the original shoreline.

(d) The first approximation of the structure's distance offshore is given by $y' = d'/\tan\beta$. The first approximation of the salient extension is then given by $y_s' = SAR\ y'$. If this value of y_s' is approximately equal to the value of y_s' originally assumed, the value is adopted. If there is a significant difference, a new estimate of y_s' is made, and the above procedures are repeated until the two values are approximately equal.

(e) The range of structure lengths as a function of nearshore wavelength for beach Type B is given by,

$$1.8\ L_{o5} < \ell < 3.0\ L_{o5} \tag{4-6}$$

and for beach Type C by,

$$1.4\ L_{o5} < \ell < 2.3\ L_{o5} \tag{4-7}$$

The range of structure lengths as a function of offshore distance for beach Type B is given by,

$$0.8\ y < \ell < 2.5\ y \tag{4-8}$$

and for beach Type C by,

$$1.0\ y < \ell < 3.5\ y \tag{4-9}$$

Applying Equations 4-6 through 4-9 gives two ranges for the breakwater length. The breakwater length adopted is the average of the highest minimum and the lowest maximum of the two ranges.

(f) If the length of the shoreline to be protected exceeds twice the breakwater length, the gap width can be selected by using the following ranges of gap width that are valid for both Type B and Type C beaches,

$$0.7\ y < b < 1.8\ y \tag{4-10}$$

and,

$$0.5\ L_{o5} < b < 1.0\ L_{o5} < \tag{4-11}$$

As above, the gap width is selected as the average of the highest minimum and the lowest maximum of the two ranges.

(g) The calculated breakwater length, gap width, distance from shore, and SAR can then be used to develop a final breakwater system design subject to subsequent evaluation using analytical tools such as computer simulations, etc.

h. Artificial headlands.

(1) Artificial headlands or headland breakwaters are constructed either on or very near the original shoreline and are within the average surf zone (Pope 1989). They are designed to form a tombolo and function as a total block to the inshore littoral transport. Beach fill is usually incorporated into the project design, since these structures are not very efficient in trapping the regional longshore transport. Downdrift effects with headland breakwaters can be significant; therefore, they should be used in areas where there is minimum net littoral transport and the downdrift areas are not considered sensitive.

(2) A definition sketch of an artificial headland breakwater was provided in Figure 4-4, with the relationship between the variables, and thus the spacing and location of the breakwaters, presented in Figure 4-9 (Silvester et al. 1980; USAED, Buffalo 1986). The relationship between the spacing and indentation and the angle was derived from measurements of natural headland embayments known to be in equilibrium. When in equilibrium, a bay will experience no littoral drift movement since the predominant waves arrive normal to the beach at all points around the periphery (Silvester 1976). With sediment supplied, the shoreline will continue to be seaward of the static equilibrium position obtained using Figures 4-4 and 4-9, and longshore transport will continue to be bypassed downdrift.

(3) Most beaches associated with headlands assume a shape related to the predominant wave approach: a curved upcoast section representing a logarithmic spiral and a long and straight downcoast section (Chew et al. 1974). The logarithmic spiral shape of the beaches associated with headlands has been investigated extensively (Silvester 1970, 1974, 1976; Chew et al. 1974; Rea and Komar 1975; Silvester et al. 1980; Everts 1983; Berenguer and Enriquez 1988; Hsu et al. 1989).

(4) At artificial headland sites subject to bidirectional wave attack, the artificial headlands may have to be shore-connected with groins to prevent breaching. Alternatively, the breakwater length can be increased (USAED, Buffalo 1986).

i. Perched beaches.

(1) Perched beaches have not been studied extensively, and very few have been built; consequently, there is little information on which to base a design.

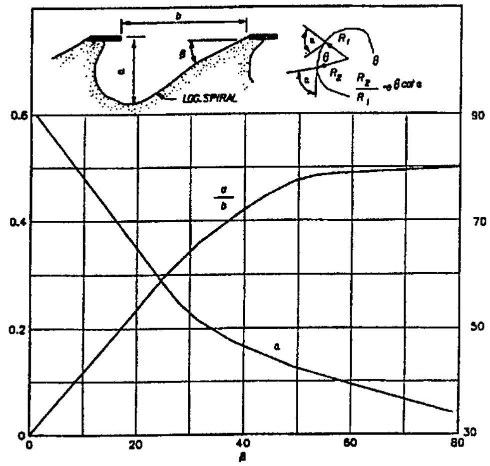


Figure 4-9. Parameters relating to bays in static equilibrium (after Silvester et al. 1980)

The concept has been investigated in the laboratory (Chatham 1972; Sorensen and Beil 1988) and in the field (Inman and Frautschy 1966; Sivard 1971; Douglass and Weggel 1987). Inman and Frautschy (1966) discuss a natural "perched beach" at Algodones in the Gulf of California; Sivard (1971) discusses a man-made perched beach at Singer Island, Florida, constructed of large, sand-filled bags. Douglass and Weggel (1987) discuss the performance of the perched beach at Slaughter Beach, Delaware, built under the Shoreline Erosion Control Demonstration Act of 1974. The sill structure used to construct a perched beach can be considered a special case of a nearshore breakwater, one with a low-crest, a high wave transmission coefficient and extending a relatively long distance along the coast. Whereas the objective of the nearshore breakwater is to shelter a section of the coast from wave action, the objective of the perched beach sill is to introduce a discontinuity into the beach profile. The profile on the landward side is at a higher elevation than the profile on the seaward side.

(2) A dimensional analysis for the design of submerged sills is located in Appendix D. As more experience with perched beaches accumulates, the preceding dimensionless terms can be used to relate the behavior of various installations to each other. Unfortunately, there is currently little experience on which to base a design.

Chapter 5
Construction and Postconstruction Activities

5-1. Objectives

a. General. The objectives of this chapter are to provide guidelines for activities to be undertaken during and following the construction of a beach stabilization project. These activities include documentation of construction records, new or unusual construction techniques, construction problems encountered and their solutions, necessary design modifications (to provide as-built information), and periodic postconstruction inspections. The requirements for the preparation of an "Operations and Maintenance Manual" for use by local sponsors in operating the project are also presented. In addition, postconstruction monitoring may be undertaken to evaluate a project's functional and structural performance with the aim of developing guidance and methodology for the design of similar type projects in the future.

b. Requirements and guidance. Specific performance requirements and guidance for accomplishing the satisfactory maintenance and operation of shore protection works, including coastal structures and beach-fill projects, are provided in ER 1110-2-2902. This regulation prescribes operations, maintenance, inspection, and record keeping procedures required to obtain the intended purposes of shore protection projects. According to ER 1110-2-2902, the Corps, while not responsible for the maintenance of shore protection projects, is involved in the periodic reconstruction or nourishment of such projects. The Federal participation is conditioned on the non-Federal interest assuring operation, maintenance, replacement, and repair of improvements during the economic life of the project as required to serve the intended purposes.

5-2. Construction Records

During construction of a beach stabilization project a daily log should be maintained by the Corps of Engineers' inspector. Items such as the construction techniques used and problems encountered along with their solutions should be noted. New, unique, and innovative construction practices should be documented along with an assessment of their success. In addition to information relevant to the project under construction, information that might be useful for the design and/or construction of future projects should be noted. A photographic history of construction with documentation giving dates and locations of the photographs and what is being illustrated should be maintained. Changes that deviate from the original design must be documented to provide an as-built record of the project. Project drawings should be marked up and revised to show the as-built conditions. It is also important that the design engineer conduct periodic site visits during project construction.

5-3. Inspections

Following construction, and for the lifetime of the project, periodic inspections of the project should be conducted. The frequency of inspection will depend on the type of project, the physical environment at the site, and the scope of the project. Annual inspections of projects involving beach fill should be made since significant beach changes can occur over a single storm season. In addition, inspections should be made following severe coastal storms. Inspections should focus on potentially dangerous conditions; conditions that can compromise the public safety, such as hazards to swimmers or navigation, must be identified so that remedial measures can be promptly taken. Structural deterioration that impairs a project's ability to function or that imperils the structure itself should be noted. Repairs that may prevent unraveling of the structure should be made in a timely manner. Photographic documentation should be provided if appropriate. Shoreline and\or bathymetric changes that may be precursors of a functional or structural failure should also be identified. For example, scour at the toe of an offshore breakwater, groin, or seawall may indicate imminent collapse and failure.

5-4. Monitoring

a. Functional performance. Monitoring the functional performance of a beach erosion control project may serve two purposes: to identify deficiencies in the performance of the project so that modifications can be made to improve its performance (operational monitoring), or to evaluate the adequacy of design methods used and, if necessary, to improve them (research monitoring). The design of beach erosion control structures is not an exact science. The marine environment is harsh; it is corrosive, abrasive, and subject to unpredictably severe and unusual storms. Even the best designs are usually based on insufficient and/or inadequate data. Annual average wave conditions and sediment transport rates can change significantly from year to year making the design of beach erosion control structures difficult. It is not unusual for projects to be modified during their lifetime to improve their performance based on observations of their behavior. Likewise, design methods for beach erosion control

projects are evolving; they are being modified and improved as experience with prototype projects is gained. Monitoring completed projects can provide the data needed to improve design guidance. These improved methods will lead to better, more cost-effective projects in the future. Each project monitoring program will need to be individually developed since each project is unique and has site-specific conditions that must be evaluated and documented. Also, the objectives of a monitoring program will differ from project to project. The following discusses several types of basic data that are often included in beach stabilization project monitoring programs. In addition to data collection, data analysis methods must be considered in the monitoring plan and entered into the monitoring budget.

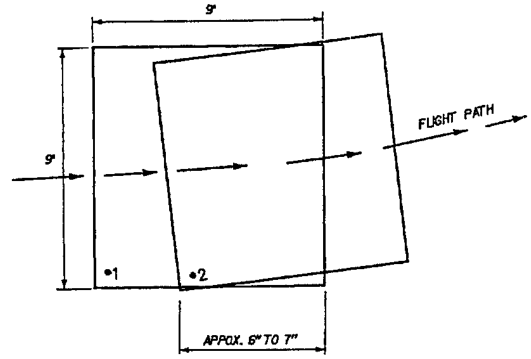


Figure 5-1. Typical 9- by 9-inch aerial photography showing 60-percent overlap (schematic)

(1) Photographic documentation. An easy and relatively inexpensive way to monitor performance of a beach erosion control project is to obtain periodic photographs of the project. An inexpensive procedure is to periodically obtain ground-level photographs of the same scene taken from the same location. While this method gives a history of project performance, it is mostly just qualitative. Quantitative data can be obtained from controlled, vertical aerial photography of a project area. Data on ground elevation, shoreline and berm location, offshore shoals, structure geometry, and deterioration can all be obtained from aerial photographs. In addition, beach-use and land-use changes can also be monitored. Aerial photographs should have an appropriate scale; a 1:4800 (1 inch = 400 feet) scale generally provides sufficient detail and is typically used for coastal project monitoring. Photographs are usually 9- by 9-inch contact prints of color or black and white negatives (Figure 5-1). Larger scales (usually enlargements of 9- by 9-inch negatives) have also been found useful for specific applications, e.g. monitoring the movement of rubble-mound structure armor between successive photographic flights. Adjacent aerial photographs should have a 60-percent overlap so that they can be analyzed stereographically to obtain ground elevations. The frequency of photography depends on the purpose of the monitoring. If the purpose is inspection, annual flights may suffice; if the purpose is detailed monitoring of project performance, quarterly, monthly, or more frequent flights may be necessary.

(2) Beach profiles and bathymetric changes.

(a) The design objective of a stabilization project is to maintain a wide beach; consequently, the best indicator of a project's success or failure is the condition of the beach. Beach profiles, obtained periodically, can document the accretion, erosion, or stability of the project's shoreline. The frequency of beach surveys depends on the objectives of the monitoring program. If the monitoring is operational or the objective is to develop design methods and/or document transient phenomena such as performance immediately following construction or poststorm recovery, quarterly, monthly, or even more frequent surveys should be conducted. The quality and detail will depend on the purpose. It is important to note that monitoring will not only assist with routine evaluation of the project but may significantly assist in documenting storm damages or damages prevented.

(b) The spacing along the beach of profile lines will also depend on monitoring objectives. If only general shoreline trends are needed, distantly spaced profiles may suffice, i.e., if one or two profiles can be assumed to describe conditions and changes occurring over a relatively long stretch of beach (Figure 5-2). In contrast, if calculations of accretion and\or erosion volumes are needed or if seasonal volume changes need to be documented, profile lines must be spaced close enough to allow reasonably accurate volume computations. At a minimum, there should be at least three profile lines within a groin compartment, spaced at the most several hundred feet apart. Similarly, there should be three or more profile lines in the lee of a detached breakwater depending on the breakwater's length, distance from shore, etc. (Figure 5-3). In some cases, subaerial profile changes will provide sufficient information. For example, if changes only in the location of the berm or the low-, mean-, or high-tide level shorelines are needed, subaerial or, at most, wading surveys will suffice. On the other

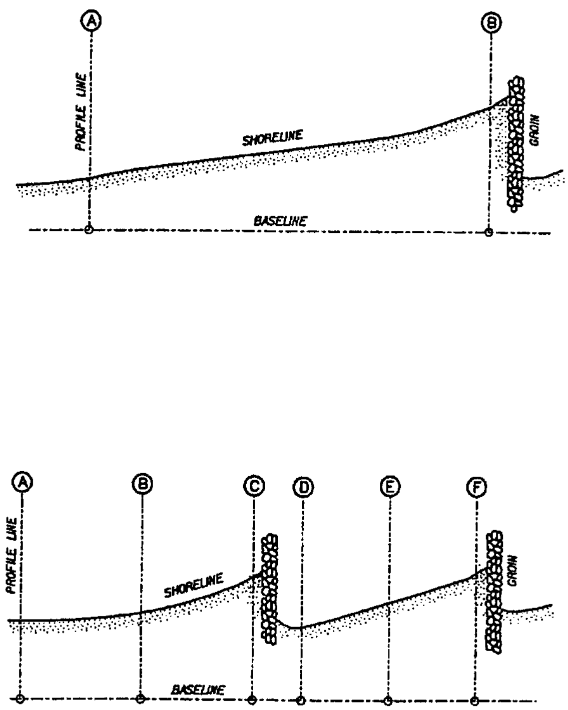


Figure 5-2. Spacing of profile lines where beach changes are gradual and severe

hand, if bathymetric changes brought about by project construction or seasonal offshore profile changes are to be documented, profile lines will have to extend offshore beyond wading depth. Cost is often a factor. Subaerial profiles are relatively inexpensive to obtain whereas profiles extending some distance offshore are more costly. The two surveys (subaerial and subaqueous) must be matched, usually in the surf zone where changes are large and where they occur quickly. Thus, subaerial and corresponding subaqueous surveys must be done within a short time of each other with no intervening storms.

(3) Wave conditions.

(a) Waves and the longshore currents they cause are the dominant sediment moving forces in the nearshore zone. Waves also cause the critical forces that act on coastal structures. Thus, wave data are needed to establish cause and effect relationships involved in the performance of beach erosion control projects.

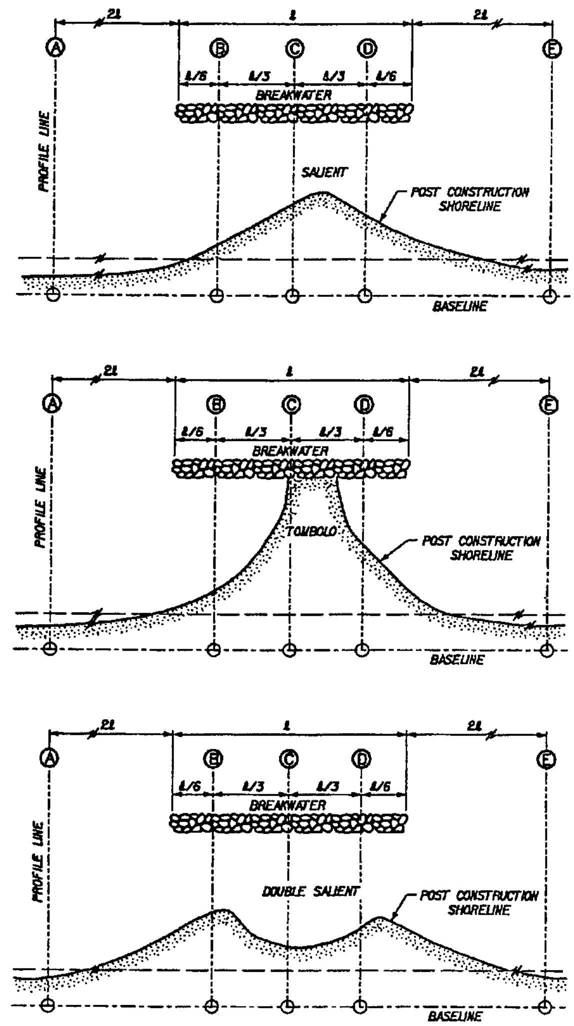


Figure 5-3. Spacing of profile lines in the lee of a detached breakwater

(b) The deployment and operation of wave height and/or wave height and direction recording instruments may be justified for more detailed research monitoring programs where the cause and effect relationships between waves, resulting sediment transport, and project performance need to be established. Various types of gages are available, selection of which will depend on the type of wave data needed and the physical conditions at the site where the gage will be used. For sediment

transport studies, a wave measuring system that provides information on wave direction is often needed. This usually requires the deployment of two or more wave gages or the measurement of water particle velocities in addition to wave height or subsurface pressure. If only wave height is needed, an accelerometer buoy, subsurface pressure gage, or surface piercing gage will suffice. Wave gage selection, installation, operation, signal processing, and data analysis usually require the assistance of qualified personnel.

(c) An inexpensive alternative to wave gages is to employ coastal observers who can make daily observations of wave, wind, and nearshore current conditions. The CERC's LEO program (Schneider 1981) is an example. Under the LEO program, volunteers make daily observations of breaking wave heights, breaker periods, and the angle that incoming wave crests make with the shoreline. In addition, they obtain data on longshore current velocity, surf zone width, foreshore beach slope, and wind speed and direction. A disadvantage is that the data depend critically on the diligence and skill of the observer. Thus, data quality varies from observer to observer and possibly from day to day for a given observer. It is often for storm conditions when wave data are critically needed that volunteer observers are unable or unwilling to make observations. The quality of some elements of the LEO data set is better than others, and careful interpretation of the data is important. Interpretations and conclusions drawn from the data must recognize the limitations of the measurements. Wave observation sites must be carefully selected to avoid locations where conditions may not be representative of an area. This is true for visual observation sites as well as for wave gage sites. On the other hand, wave measurements may be desired in sheltered areas for some applications; for example, wave measurements behind a nearshore breakwater will determine wave attenuation characteristics of the structure.

b. Structural performance. Structural performance and functional performance are closely related. When a project fails structurally, it often loses its ability to function. The extent of a structural failure determines the extent of any associated functional failure. Some structures, such as rubble-mound structures, can experience a certain level of damage without total loss of functioning ability. These structures fail progressively rather than catastrophically, and they are considered "flexible." Other structures, such as bulkheads and seawalls, cease to function following a structural failure. The failure of this type of structure is more rapid than progressive, and they are considered "rigid." Like functional performance monitoring, two types or levels of structural monitoring can be undertaken. Structural monitoring can be performed to simply establish if a given structure has sustained damage so that repairs might be made in a timely manner (operational monitoring), or it can be performed to obtain data to improve design methods (research monitoring). Operational monitoring might involve only little more than periodic inspections, whereas research monitoring might involve more elaborate wave and wave force measurements.

(1) Photographic documentation.

(a) A relatively inexpensive way to document structural performance is to periodically inspect the structure and photograph areas of structural deterioration. Photographs should be accompanied with a written description of the damage, an indication of where on the structure the damage is located, and its probable cause. The location can be indicated on appropriate project drawings and\or on a project map. Aerial photographs can also be used to get an overall picture of structural damage, particularly damage surveys of rubble-mound structures following major storms. Aerial surveys have the added advantage of affording access to what might otherwise be an inaccessible area.

(b) For more detailed research monitoring, controlled, vertical, aerial photographs can be used to obtain quantitative data on rubble armor unit movement or on the lateral displacement of other structures. Large-scale stereographic pairs of photographs can provide information on changes in the elevation of structural components, such as armor units. A reference set of photographs taken shortly after construction can serve as a base condition against which subsequent photographs can be compared.

(2) Wave conditions.

(a) Data on wave conditions are needed to determine the conditions under which structural damage or failure may have occurred or to correlate with wave force measurements. Recorded wave data, however, are generally not obtained under routine operational structure monitoring because of the high cost of obtaining it. Rigid structures such as sheet-pile groins are usually designed for high waves in the spectrum (the l-percent wave or higher), and design wave conditions are selected with a return period of many years. Unless wave conditions exceed design conditions, damage and failure will probably not occur.

(b) Rubble-mound structures are designed for lower waves in the spectrum (usually the 10-percent or significant wave) and for wave conditions with a relatively low return period since they can sustain some damage without failing functionally. Wave gages are sometimes deployed for the research monitoring of rubble-mound structures. Large waves associated with storms are of primary interest since they result in armor unit displacement and other damage. Wave direction information is usually of secondary importance for structural monitoring, and a single wave buoy, subsurface pressure gage and\or surface piercing gage usually suffices.

(c) If they can be obtained, LEO data can provide an inexpensive alternative source of wave information. Data are usually needed for storm waves, and it may not be possible for an observer to obtain wave height estimates under storm conditions. If data can be obtained, their accuracy may be suspect.

(3) Wave force measurements. Wave force and\or pressure measurements on rigid beach erosion control structures may be desired for research monitoring purposes. In conjunction with the force or pressure measurements, wave height data at the structure would have to be obtained to develop correlations. Wave force or pressure data, however, are not usually obtained under routine monitoring.

5-5. Operations and Maintenance Manual for Local Sponsors

a. Requirements. ER 1110-2-2902, "Prescribed Procedures for the Maintenance and Operation of Shore Protection Works," requires that an Operation and Maintenance (O&M) manual be prepared for local sponsors of federally constructed shore protection works who are responsible for operating and maintaining such projects. The Federal government, however, must provide local sponsors with an O&M manual containing guidance on how to operate the project in a way to achieve project objectives. This responsibility requires that a certain level of project monitoring be undertaken to obtain data on which operational decisions can be made. The local sponsor must identify a "superintendent" in charge of operating the project who must prepare an emergency plan to respond to storms exceeding the project's design storm so as to minimize any threat to life and property. He will maintain organized records on the operations, maintenance and repair, condition, inspection, and replacement of the project's elements including any structures and beach fills. The O&M manual and, therefore, any operations monitoring plan should address the four elements of a shore protection project: the beach berm and foreshore, the protective dune, coastal structures, and any appurtenant facilities. The monitoring requirements of ER 1110-2-2902 should be viewed as minimum monitoring requirements.

b. Poststorm condition surveys. Regarding coastal structures, ER 1110-2-2902 requires that poststorm condition surveys be made of any structures to include the identification of seepage areas, piping or scour beneath or through the structures, settlement that might affect stability, condition of the materials of which the structure is built, identifying conditions such as concrete spalling, steel corrosion, encroachment on the structure that might endanger the structure or affect its functional performance, accumulation of trash and debris; bank scour; toe erosion; flanking erosion; drainage systems; the condition of any mechanical/electrical systems such as pumps, navigation lights, etc.; and assurance that no boats or floating plant are tied up to the structures.

Appendix A
References

A-1. Required Publications

ER 1110-2-1407
Hydraulic Design for Coastal Shore Protection Projects

ER 1110-2-2902
Prescribed Procedures for the Maintenance and Operation of Shore Protection Works

EM 1110-2-103
Construction of Shore Protection Works

EM 1110-2-1204
Environmental Engineering for Coastal Protection

EM 1110-2-1412
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EM 1110-2-1414
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EM 1110-2-1601
Hydraulic Design of Flood Control Channels (Chapters 1 through 4)

EM 1110-2-1612
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EM 1110-2-1616
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EM 1110-2-1903
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EM 1110-2-2502
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Appendix B
Advantages and Disadvantages of Various Beach Stabilization Structures

B-1. Groins

a. Advantages.

(1) Groins are effective against erosion caused by sand losses due to longshore transport.

(2) A wealth of data is available on the performance of groins in various physical environments.

(3) Groins can be built using shore-based equipment and are therefore often less expensive to construct.

(4) Groins do not change the character of the surf zone. Wave heights along a beach after groin construction are virtually unchanged.

(5) Groins can be constructed of various types of materials, e.g., rubble-mound, steel, and concrete sheet piling, timber, etc.

(6) By adjusting their dimensions and permeability, groins can be designed to either completely block longshore transport along the beach face or to allow sand bypassing.

b. Disadvantages.

(1) Groins are not effective in preventing offshore sand losses.

(2) Groins can cause rip currents to develop along their flanks and thus might enhance offshore sand loss.

(3) Groins may starve downdrift beaches of sand if they do not allow bypassing.

(4) There is a range of conflicting design philosophies: permeable versus sand tight; high versus low; long versus short, etc.

B-2. Detached breakwaters

b. Advantages.

(1) Detached breakwaters are effective against erosion caused by both alongshore and offshore sand losses.

(2) Detached breakwaters have been proven to stabilize shorelines.

(3) Detached breakwaters are often aesthetically acceptable when other shore stabilization structures are not. (They can be designed to be submerged over most of a tidal cycle.)

(4) They can be built of inexpensive, readily available materials, e.g., rubble-mound, dumped stone, etc.

(5) They can be built to allow sand bypassing and control the rate of bypassing.

(6) They can be designed to permit overtopping to improve water quality in the breakwater's lee.

(7) There is extensive foreign experience in using nearshore breakwaters for shoreline stabilization.

(8) Nearshore breakwaters can significantly reduce wave heights along a reach of shoreline.

b. Disadvantages.

(1) Detached breakwaters may be expensive to construct because they are not connected to shore and may require either temporary structures or floating plant to support construction equipment.

(2) Breakwaters significantly alter the character of the surf zone and may restrict certain beach activities, e.g. bathing in the vicinity of the structures, surfing, etc.

(3) They may pose a navigation hazard and may require the installation and continued maintenance of aids to navigation.

(4) They may pose a hazard to swimmers.

(5) If improperly designed, they could cause water quality problems due to poor circulation behind them.

(6) There has not been extensive experience in using nearshore breakwaters for shoreline stabilization in the United States.

(7) Detached breakwaters may connect with shore by forming a tombolo. This could seriously interrupt longshore transport and cause downdrift erosion.

B-3. Artificial Headlands

a. Advantages.

(1) The installation of artificial headlands or headland breakwaters can produce a stable shoreline similar to the stable pocket beaches observed with natural headlands.

(2) Although a relatively new practice, it has been applied successfully in numerous countries.

b. Disadvantages. Downdrift effects with headlands can be significant and continuing.

B-4. Submerged Sills

a. Advantages.

(1) Submerged sills (perched beaches) may be more aesthetically acceptable than groins or breakwaters because they are usually submerged and not visible from shore.

(2) Submerged sills reduce the level of wave action on a beach.

(3) They slow/retard offshore sand losses from a beach.

b. Disadvantages.

(1) The low sill structure may not be high enough to significantly reduce wave action and may not retard offshore losses.

(2) The submerged sill may prevent beach recovery during beach-building wave conditions.

(3) Submerged perched beach structures may pose a hazard to navigation.

(4) There has not been much experience with submerged sills/perched beaches; therefore, there are not much data upon which to base a design.

(5) It may be difficult and expensive to build the sill structure because it is both offshore and submerged. Construction may require floating plant and thus may be expensive.

(6) The submerged sill may be difficult to inspect since it is underwater.

B-5. Alternative Shoreline Stabilization Devices and Methods

a. Advantages.

(1) Alternative shoreline stabilization devices and methods may have the potential of being more effective and cheaper than traditional shoreline stabilization methods.

(2) They could be proposed and built as experimental projects and subsequently modified as needed to gain experience.

b. Disadvantages.

(1) Most alternative shoreline stabilization methods are virtually untried, and there is little information available on their performance; consequently, there is little information on which to base a design.

(2) A costly, major experimental/developmental program would have to be undertaken to obtain information on which to base a design. This might involve both laboratory and prototype studies.

(3) Operations and maintenance costs are unknown because of the lack of long-term experience.

(4) An alternative shoreline stabilization method, like any stabilization system, would have to be justified economically by the savings realized through increasing the time between periodic renourishments. Data to economically justify alternative methods are generally not available.

Appendix C
Dimensional Analysis for Groin Design and Example Applications

C-1. Dimensional Analysis.

A dimensional analysis of the variables important in groin design can provide insight into the factors governing the functional design of groins. The variables that describe the behavior of groins are summarized in Figure C-1. For a groin system, the variables and their dimensions (in square brackets) are,

ℓ_u = groin length along the updrift side of the groin (measured from the beach berm), [L]

ℓ_d = groin length along the downdrift side of the groin (measured from the beach berm), [L]

x = distance between adjacent groins (groin spacing), [L]

H_b = breaking wave height in the groin compartment, [L]

d_b = water depth in which waves break, [L]

d = water depth at the seaward end of the groin, [L]

T = wave period, [T]

z = mean tidal range at site, [L]

α_o = original shoreline orientation (measured from some arbitrary alignment), [dimensionless]

α = reoriented shoreline alignment, [dimensionless]

or, in lieu of the above two angles

$\delta\alpha = \alpha_o - \alpha$ = change in shoreline alignment caused by groins, [dimensionless]

Note: $\delta\alpha = \tan^{-1}\left[\frac{\ell_d - \ell_u}{x}\right]$

g = acceleration of gravity, $[L]/[T]^2$

Q_n = original potential net longshore sand transport rate $[L]^3/[T]$

Q_{groin} = potential longshore transport rate with groins, $[L]^3/[T]$,

A_s = "wet area" between groins, $[L]^2$,

Note: $A_s \approx \left[\frac{\ell_d + \ell_u}{2}\right] x$

K_r = reflection coefficient for the groin, [dimensionless]

K_t = transmission coefficient, [dimensionless]

h = groin height above the mean low water (MLW) line, [L]

One possible set of dimensionless variables is given by,

$\pi 1 = \frac{x}{\ell_u}$ = dimensionless groin spacing based on updrift groin length

$\pi 2 = \frac{x}{\ell_d}$ = dimensionless groin spacing based on downdrift groin length

or, $\frac{\ell_u}{\ell_d}$ = shoreline offset across a groin

can be substituted for either of the two preceding dimensionless terms,

$\pi 3 = \frac{H_b}{d}$ = dimensionless breaker height (a measure of where breaking waves occur relative to the groin's end)

$\pi 4 = \frac{d}{d_b}$ = dimensionless water depth at the groin's end at MLW

$\pi 5 = \frac{d}{\ell_u}$ = average beach slope along the updrift side of the groin

or alternatively, $\pi' 5 = \frac{d}{\ell_d}$ = average beach slope along the downdrift side of the groin

$\pi 6 = \delta\alpha = \tan^{-1}\left[\frac{\ell_d - \ell_u}{x}\right]$ shoreline reorientation

$\pi 7 = \frac{H_b}{gT^2}$ = dimensionless breaker height index, (proportional to H/L)

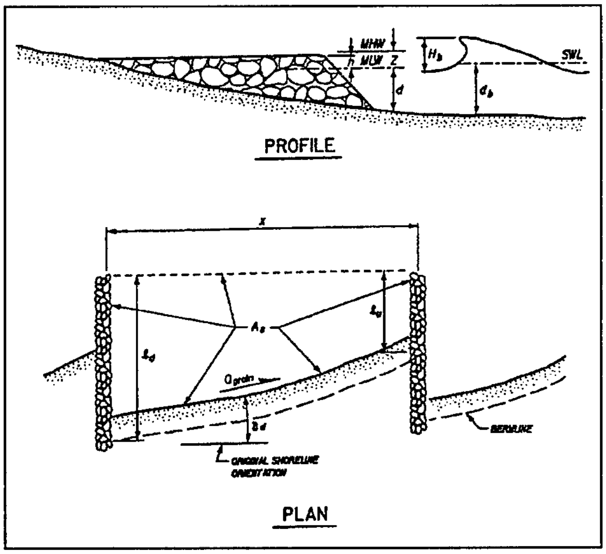


Figure C-1. Definition of terms--dimensional analysis of groins and groin fields (MHW = mean high water; MLW = mean low water; and SWL = still-water level)

$\pi 8 = \dfrac{A_s}{xd}$ = dimensionless water area between two adjacent groins,

$\pi 9 = \dfrac{z}{d}$ = dimensionless tidal range (based on at end of groin)

$\pi 10 = \dfrac{h}{z}$ = dimensionless groin crest height,

Note: If $h/z \geq 1$, then the groin crest is above the MHW line; if $0 \leq h/z \leq 1$, the groin crest is within the mean tidal range, and if $h/z \leq 0$, the groin is submerged at low tide.

$\pi 11 = K_r$ = wave reflection coefficient for the groin

$\pi 12 = K_t$ = wave transmission coefficient for the groin

$\pi 13 = \dfrac{Q_n}{H_b^{5/2}\, g^{1/2}}$ = a dimensionless measure of the net longshore transport

$\pi 14 = \dfrac{Q_{groin}}{Q_n}$ = dimensionless net longshore transport reduction attributable to the groins.

a. π1 and π2 are dimensionless groin spacings, one based on groin length as measured along the updrift side of a groin, the other based on length measured along the downdrift side. Rule 7 (refer to Chapter 3) suggests that $\pi 1 \leq 3$ and $\pi 2 \geq 2$.

b. π3 determines whether waves normally break seaward or landward of the groin's end. If $\pi 3 \geq$ about 0.78, waves will generally break seaward of the groin's end, and sand will bypass the groin even during normal wave conditions. If $\pi 3 \leq$ about 0.78, waves will normally break landward of the groin's seaward end. The magnitude of π3 determines whether a groin is "long" or "short." Similarly, π4 determines whether waves break seaward or landward of the groin's end.

c. The average beach profile slope along the updrift side of the groin is indicated by π5.

d. π6 is a measure of the reorientation of the shoreline between two adjacent groins in a groin system or a measure of the shoreline discontinuity between the two sides of a groin.

e. π7 is an indicator of the wave environment at the site. The assumption here is that the wave environment can be described by a single "characteristic wave." The "characteristic wave" is one that best describes longshore sand transport conditions at the site. The mean wave height and mean wave period at a site might be used as the "characteristic wave."

f. π8 is a measure of that area between two adjacent groins in a groin system that will not fill with sand or will not retain sand. It is a measure of how much sand will be removed from the groin system before the shoreline reaches a quasi-steady equilibrium configuration. If the shoreline between two groins is assumed to be straight, π8 is related to π1, π2 ,and π5 approximately by $\pi 8 = \frac{1}{2}\, \pi 1 (\pi 1 + \pi 2)/\pi 5$.

g. The dimensionless tidal range is given by π9. It is a measure of how much the water depth at the end of the groin changes over a tidal cycle.

h. The height above the groin crest above the MLW (or mean lower low water) line is given by π10. If $\pi 10 \geq 1$, then the groin crest is always above the water level except possibly during storm surges. If $0 < \pi 10 < 1$, the water level is alternatively above and below the groin's crest depending on the stage of the tide. For $\pi 10 \leq 0$, the groin is submerged even at low tide.

i. π11 and π12 are the wave reflection and wave transmission coefficients of the groin. They are probably of lesser importance to the successful function of groin

systems than are the other dimensionless variables.

j. $\pi13$ and $\pi14$ are measures of the longshore transport environment at the groin site. $\pi13$ is a measure of the potential net transport and, indirectly, a measure of a characteristic longshore transport wave angle. $\pi14$ is a measure of the reduction in longshore transport brought about by the construction of groins. It is indirectly a measure of how much the shoreline is reoriented--and how much sand transport is reduced--by the groins.

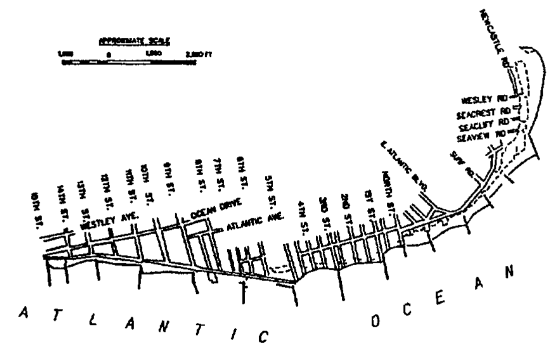
C-2. Example Applications.

Many of these dimensionless variables can be determined from an analysis of nearby groins and transposed to the site of a proposed groin project. Analysis of aerial photographs and field measurements can be used to determine reasonable values for the above variables. They can then be used to functionally design a groin system. Even data taken from groin projects deemed to be unsuccessful can be examined in light of the foregoing dimensionless terms and modified to develop a successful design. Application of the dimensionless parameters to a groin design is illustrated in Example 1.

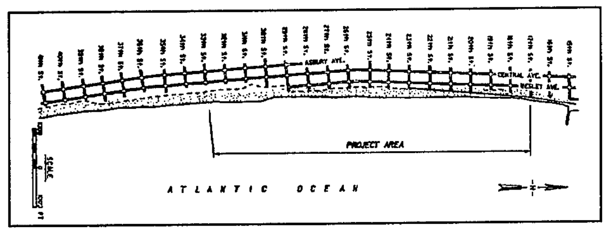
a. Example 1

(1) Problem: (The example that follows is entirely hypothetical and not intended to be an actual design.) A beach fill is planned for an 18-block-long area of Ocean City, NJ, from 17th Street in the north to 33rd Street in the south. See Figures C-2a, b, and c for a location map. A minimum beach berm width of 100 feet* measured seaward from the existing bulkhead line is desired. Groins are to be evaluated as a means of stabilizing the beach and retaining the fill within the project area. Because of the developed nature of the shoreline, the potential for erosion along both updrift and downdrift beaches is a concern. Some of the fill is expected to leave the project area to nourish adjacent beaches.

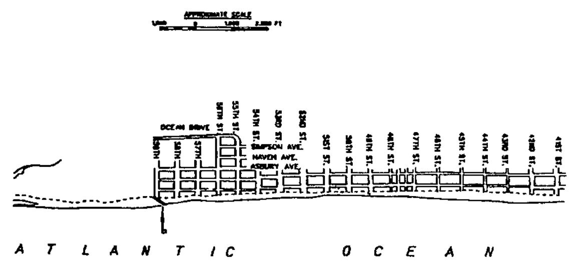
(2) Groins have been built along adjacent beaches to stabilize them. North of about 15th Street, groins are located about 1,000 feet apart along the shoreline. Also, there is a "terminal" groin at the south end of the developed portion of the island at 54th Street. Analysis of aerial photographs taken of these other groins indicates that the shoreline alignment varies from an average azimuth of about 37 degrees to about 70 degrees.



a. Northern section of Peck Beach Project area



b. Center section of Peck Beach Project area



c. Southern section of Peck Beach Project area

Figure C-2. Location map for Example 1, Ocean City, NJ, shoreline

Table C-1 gives the shoreline azimuth taken from three sets of aerial photographs for both high and low tide shorelines within the various groin compartments. Along the northerly Ocean City beaches, the shoreline alignment in the groin compartments reflects the variation in potential longshore transport rates and directions caused by the proximity of Great Egg Harbor Inlet and its offshore shoals. The net longshore sediment transport is southward along most of Ocean City's shoreline; however, along the northernmost beaches, due to sheltering of the beaches from waves out of the northeast by the shoals offshore of Great Egg Harbor Inlet, the net transport is

* To convert feet into meters, multiply by 0.3048

Table C-1
Shoreline Orientation at Various Locations in Ocean City, NJ

Location	Shoreline Orientation*					Average
	25 October 1965		30 March 1984	8 August 1984		
	HWL**	LWL	LWL	HWL	LWL	
34th Street	55.50	55.05		55.50	55.50	55.50
25th Street				58.50	58.50	58.50
15th Street						
13th Street	63.50	62.50	63.00	64.50	69.00	64.40
11th Street	64.25	62.00	73.50	74.00	71.00	68.95
9th Street	58.25	73.50	71.00	72.50	75.00	70.05
7th Street		60.00		76.00	70.00	68.67
5th Street	48.00	65.50		65.00	64.00	63.13
3rd Street	60.75	62.00	60.00	61.50	76.50	64.15
1st Street	56.50	64.00		50.00	52.00	53.63
North Street	60.50	61.00	59.50	63.00	64.00	61.60
Groin E	54.50	59.50	44.00	59.00	60.00	55.40
Groin D	46.50	51.00	41.00	43.00	43.00	44.90
Groin C	40.00	47.00	30.00	34.50	32.50	36.80
Groin B	39.50	45.00	34.00	42.50	43.00	40.80
Groin A						

* Angle measured clockwise from north.
** HWL = high-water level; LWL = low-water level.

northward--toward the inlet. Thus, waves from the southeast can move sand northward whereas waves from the northeast have their sand-moving capability reduced by the inlet's ebb-tidal shoal. The result is a northward net transport along the beaches close to the inlet. Thus, the groins near the inlet are not illustrative of groins that might be built farther south.

(3) The results of an analysis of groin length taken from two sets of aerial photographs dated 25 October 1965 and 8 August 1984 are given in Table C-2. (Under actual design practice, additional sets of more recent aerial photographs that might show seasonal shoreline fluctuations would be analyzed.) The length of the groin measured from the seaward end to the high- and low-water lines along both the updrift and downdrift sides is given in Table C-2. Average values are also given. The variability in length is apparent in the tabulated values. In fact, the groins closest to the project site are those at 15th, 13th, and 11th Streets, and average values for these groins were analyzed to determine groin dimensions. Based on this analysis, the average distance from the seaward end of the groin to the low-water line along its updrift side is 208 feet; the distance to the low-water line along its downdrift side is 289 feet. The distance from the end of the groin to the high-water line along its updrift side is 396 feet, and the distance along its downdrift side is 475 feet.

(4) Typical beach profiles taken at 27th and 36th Streets are shown in Figures C-3 and C-4, respectively. Based on an average groin length of 650 feet measuredfrom the bermline, the seaward end of the groin terminates in either 4 or 6 feet of water based on these profiles. Note that these profiles are located some distance from the existing groins. In actual practice, beach profiles taken adjacent to the groins should be obtained, and the water depth at the seaward end of the groins determined. For the present example, the water depth at the end of the groin will be assumed to be 5 feet. The MLW and (MHW) lines are also shown on the profiles. The average beach slope across the intertidal zone is about 0.021.

(5) Wave conditions at Ocean City, NJ, were obtained from the Wave Information Study (WIS) hindcasts (Jensen 1983*) and compared with data presented in Table 4-4 in the *Shore Protection Manual* (SPM) (1984). Weighted average wave heights and periods were determined from the WIS hindcasts at Station 62 (Peck Beach, NJ). The weight factor was the duration that waves of a given height class or period class prevailed. Based on this analysis, the average wave height at Ocean City is 2.1 feet, and the average wave period is 6.5 seconds. The WIS wave height is in water 10 meters (32.8 feet) deep. A linear shoaling analysis to determine the breaking height of the average wave yields a breaker height of 3.0 feet. (The nearshore breaking criterion used was a ratio of wave height to water depth of 0.78.) The corresponding water depth in which the average wave breaks is thus 3.9 feet. Table 4-4 of the SPM gives an average breaking wave height of 2.8 feet at Atlantic City just north of Ocean City, and a period of 8.3 seconds. Wave heights of 2.4 and 1.8 feet and periods of 6.1 and 6.6 seconds are given for Brigantine, NJ, and Ludlam Island, NJ, respectively. These values are based on visual observations. Thus, the values determined from the WIS data and SPM (visual observation data) are in approximate agreement.

(6) The water area enclosed within the compartment formed between two groins was determined by planimetering the aerial photographs. Specifically, the compartments between the 15th and 13th Street groins and between the 13th and 11th Street groins were investigated. The areas seaward of the mean low-water line and seaward of the high-water line were determined. The values are given in Table C-3.

(7) The dimensionless variables describing the groins and groin compartments can be determined from the above variables. Note that some of the variables can be defined either for the individual groins or for the compartment formed by an updrift groin and a downdrift groin. For example, ℓ_u and ℓ_d can be defined as the distances on the opposite side of a single groin, or they can be defined as the distance measured along the updrift groin and downdrift groin at opposite ends of a groin compartment. Therefore, $\pi 1$ and $\pi 2$ are defined only for groin compartments while ℓ_u/ℓ_d can be defined either for a single groin or for a groin compartment. $\pi 1 = x/\ell_u = 1178/412.5 = 2.85$ where the distance is measured from the high-water line in the 15th-13th Street groin compartment rather than the beach berm. If the distance is measured from the low water line in the 15th-13th Street groin compartment, $\pi 1 = 4.60$. Similar analysis of the 13th to 11th Street groin compartment yields $\pi 1 = 2.43$ for the high-water line and $\pi 1 = 5.22$ for the low-water line. $\pi 3 = H/d = 3.0/5.0 = 0.6$. Since $\pi 3 < 0.78$ waves normally break seaward of the groin's end at low tide. Similarly, $\pi 4 = d/d_b = 5.0/3.9 = 1.28$. Since $\pi 4 > 1.0$, the depth at the end of the groin is shallower than the

* See References at the end of the main text.

Table C-2
Distance from Seaward End of Groins to the High-Water and Low-Water Shorelines at Ocean City, NJ, Aerial Photograph Analysis

		Groin Analysis - Ocean City, NJ							
		25 October 1965				8 August 1984			
Groin	Spacing	Length HWL, ft (Updrift)	Length HWL, ft (Down)	Length LWL, ft (Updrift)	Length LWL, ft (Down)	Length HWL, ft (Updrift)	Length HWL, ft (Down)	Length LWL, ft (Updrift)	Length LWL, ft (Down)
15th Street		465.00	504.00	323.00	387.00	360.00	480.00	280.00	340.00
	1176								
13th Street		426.00	504.00	118.00	194.00	340.00	420.00	240.00	240.00
	930								
11th Street		465.00	527.00	139.00	295.00	320.00	440.00	240.00	280.00
	1451								
9th Street		310.00	543.00	124.00	465.00	200.00	420.00	100.00	300.00
	1002								
7th Street		543.00	543.00	310.00	310.00	400.00	440.00	340.00	380.00
	1198								
5th Street		349.00	713.00	194.00	464.00			380.00	380.00
	1116								
3rd Street		658.00	658.00	349.00	388.00	600.00	600.00	440.00	440.00
	1063								
1st Street		426.00	589.00	194.00	348.00	360.00	5440.00	200.00	480.00
	534								
North Street		201.00	233.00	0	0	200.00	200.00	128.00	160.00
	690								
Groin E		349.00	504.00	78.00	271.00	280.00	340.00	280.00	260.00
	819								
Groin D		310.00	310.00	82.00	78.00	200.00	220.00	160.00	160.00
	765								
Groin C		116.00	310.00	39.00	116.00	120.00	240.00	80.00	200.00
	679								
Groin B		78.00	232.00	0	465.00	80.00	208.00	60.00	140.00
	806								
Groin A		78.00	310.00	0	387.00	80.00	520.00	60.00	380.00
AVERAGE		341.00	462.88	131.21	297.79	270.77	388.31	212.00	295.71
STD DEV		174.78	155.62	112.85	149.91	149.13	134.87	121.27	108.47

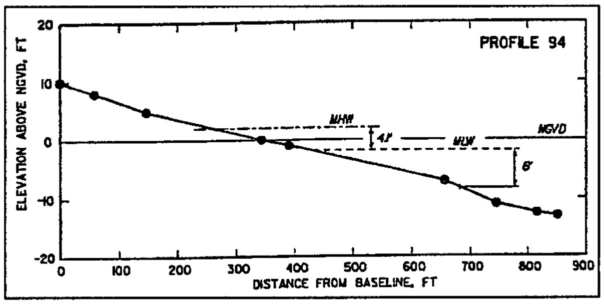


Figure C-3. Beach profile at 27th Street, Ocean City, NJ (elevation (el) measured in feet, National Geodetic Vertical Datum (NGVD)

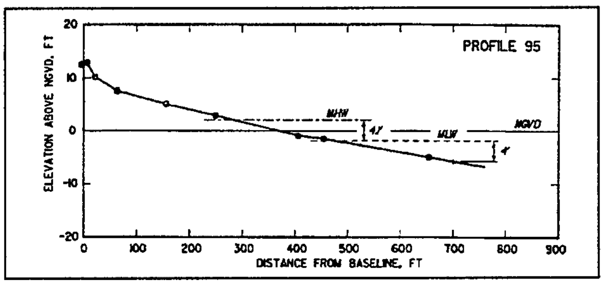


Figure C-4. Beach profile at 36th Street, Ocean City, NJ

Table C-3. Water Area Enclosed Between Groins Seaward of High and Low Water Lines

G roin Compartment	Water Line	Area (sq ft)*
15th - 13th Street	HWL	515,100
15th - 13th Street	LWL	278,600
13th - 11th Street	HWL	402,700
13th - 11th Street	LWL	216,200

* To convert square feet into square meters, multiply by 0.0929.

breaking depth at low tide. $\pi 5 = d/lu = 5.0/400 = 0.0125$, which is the average beach slope between the groin's seaward end and the high-water line along the groins updrift side, or $\pi 5 = d/l_u = 5.0/225 = 0.0222$, which is the average slope between the groin's end and the low-water line along the updrift side. Similar calculations could be made for the downdrift side of the groin. (Note these calculations are averages for the two compartments formed by the 15th and 13th Street groins and by the 13th and 11th Street groins.) $\pi 6$ cannot be determined in the present analysis since the original shoreline orientation prior to groin construction is not known so the change in orientation brought about by groin construction cannot be determined. $\pi 7 = H/gT^2 = 3/(32.17)(6.5)^2 = 0.0022$. $\pi 8 = A_s/x_d = 515{,}100/(1178 \times 5) = 87.4$ based on the MHW value of A_s and $\pi 8 = 47.3$ based on the MLW value of As. These values are for the 15th-13th Street groin compartment. $\pi 8 = 86.6$ and 46.5 for the MHW and MLW values respectively, for the 13th-11th Street groin compartment. $\pi 9 = z/d = 4.1/5 = 0.82$, the dimensionless tidal range. The existing groin crests at 15th, 13th, and 11th Streets are well above the MHW line. If the proposed groins are to be built to a crest height of MHW at their seaward end, the value of $\pi 10 = h/z$ will be $4.1/4.1 = 1.0$.

(8) Potential longshore sand transport rates at Peck Beach (Ocean City, NJ) were computed using WIS hindcast data (Jensen 1983). The analysis resulted in an annual net longshore transport rate of about 73,000 cubic yards**/year or 0.062 cubic feet†/second and a gross transport rate of 1,485,000 cubic yards/year or 1.271 cubic feet/second. Therefore, $\pi 13 = 0.00070$. Since the reorientation of the shoreline cannot be estimated, the change in longshore transport and thus $\pi 14$ cannot be calculated. However, the ratio of net transport to gross transport is $0.062/1.270 = 0.049$. The net transport represents only about 5 percent of the gross transport.

(9) Using the results of the dimensional analysis and the analysis of conditions within the two groin compartments between 15th and 13th Streets and between 13th and 11th Streets, the conditions that can be expected to prevail at the project area can be determined. The groin length and expected shoreline reorientation are shown in Figure C-5. The seaward end of the groins should be about 670 feet from the desired bermline. The high-water line along the downdrift side of the updrift groin will be about 475 feet from the end of the groin while the low-water line will be about 290 feet from the end of the groin. The resulting beach slope between the high- and low-water lines will be $4.1/(475-290) = 0.022$, which is close to the values on the existing profiles shown in Figures C-3 and C-4. The high-water line along the updrift side of the downdrift groin will be about 396 feet from the groin's seaward end while the low-water line will be about 208 feet. The beach slope between the

** To convert cubic yards to cubic meters, multiply by 0.7646).

† To convert cubic feet to cubic meters, multiply by 0.0283.

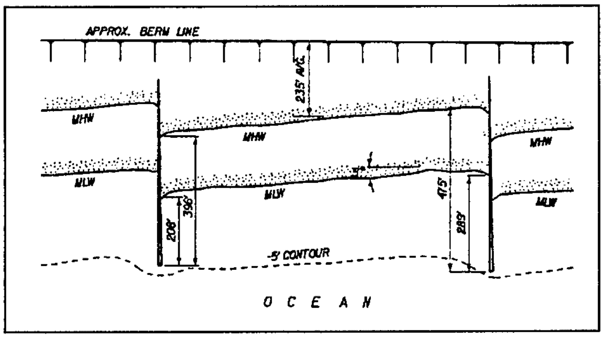


Figure C-5. Expected location of high-water and low-water shorelines after the construction of groins at Ocean City, NJ

high- and low-water lines will be 4.1/(396-208) = 0.022, or the same as along the downdrift side. The groin profile and updrift and downdrift beach profiles are shown in Figure C-6. The sloping portion of the groin has a slope of 0.022 to act as a template for the updrift beach profile. Both the updrift and downdrift profiles meet at the seaward end of the groin at about the -5-foot contour. The berm height is estimated to be at about +8 feet above mean sea level. (The approximate elevation above which dune vegetation can be established in Ocean City is about +7.5 feet.)

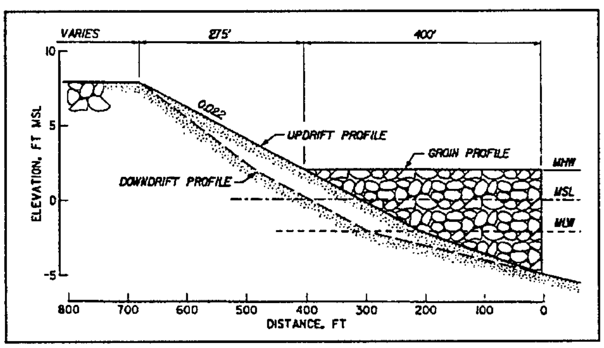


Figure C-6. Groin profile and expected beach profiles after the construction of groins at Ocean City, NJ

(10) The existing groins in Ocean City are spaced about 1,000 feet apart (Table C-2). This spacing was no doubt dictated by the spacing of the streets in Ocean Citywith the groins positioned at the ends of the odd numbered streets. Calculations of the ratio of the groin spacing to the groin length as measured from the berm line give x/l = 1,000/670 = 1.49. Also, the groin dimensions determined from the analysis give a shoreline reorientation of about 5 degrees.

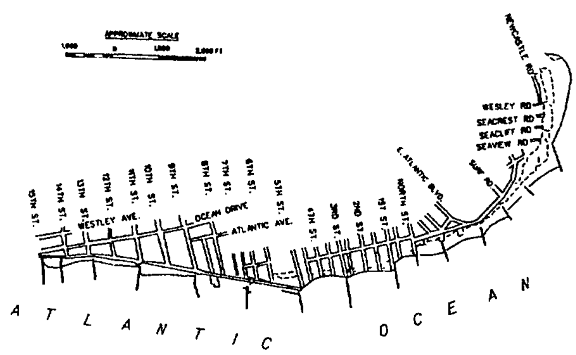
(11) Comparing the values of shoreline alignments in Table C-1, there is about a 6 degree difference in orientation between the shoreline at 25th Street and the alignment in the 15th-13th Street groin compartment. The computed values are therefore reasonable. At this point in the design, a detailed evaluation of the preliminary design might be undertaken using GENESIS (Hanson and Kraus 1989) to compute the shoreline response to a wave climate typical of Ocean City. Refinements in groin length and spacing would result.

(12) Because of the shoreline development both updrift and downdrift of the proposed project, transition sections with groins of decreasing length should be considered. Equations 3-1 and 3-2 in the main text establish the length of the groins in the transition section and their spacing. The ratio of groin spacing to length in the groin field is about 1.5; thus Equation 3-1 gives L = 0.48 L.

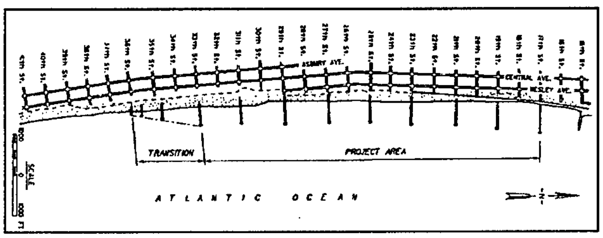
(13) Therefore, in the downdrift transition, each successive groin will be about half the length of the one updrift of it. The first groin in the transition section will be 0.48(670) = 322 feet long as measured from the desired bermline. The third will be 154 feet long, etc. The spacing given by Equation 3-2 yields S = 1.39 L. Therefore, the first groin in the downdrift transition section will be located 1.39(670) = 930 feet downdrift of the project groin field. The second will be 1.39(0.48 × 670) = 450 feet downdrift, and the third, 215 feet downdrift. The groin field and transition sections are shown in Figures C-7a, b, and c.

b. Example 2

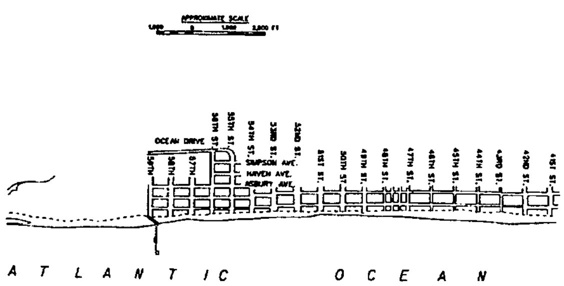
(1) The following example application of a groin design was taken from the General Design Memorandum (GDM) for a shore protection project in Manatee County Florida (US Army Engineer District (USAED), Jacksonville 1990). The authorized project consists of restoration of 3.2 miles (5.15 kilometers) of gulf shoreline on Anna Maria Key to an elevation of 6 feet above MLW, with a 50-foot berm width and natural slopes seaward as would be shaped by wave action. In addition to the initial fill, the authorized project also provided for future nourishment of the restored beach and adjacent shorelines as needed. One of the alternatives considered in the GDM was the use of groins to hold the project design cross section in front of two designated locations of shoreline. Otherwise, higher nourishment quantities would be required due to significant losses of material from these two areas.



a. Northern end of Ocean City, NJ



b. Central section of Ocean City, NJ



c. Southern end of Ocean City, NJ

Figure C-7. Location of groins and groin transition sections

(2) The following is an excerpt of the groin design section included in the Appendix of the GDM (USAED, Jacksonville 1990). It is included to show a summary of the design process that may be used in a groin project. Also included in the same appendix are summaries of the coastal parameters and natural forces such as winds, waves, currents, storm history, and shoreline change history.

c. Effects of groins on adjacent shoreline. Once the location of the two groins was determined, the next step in the design was to determine the length of the structures and the associated effective updrift and downdrift distances. Based on the performance of the pier at the Manatee County public beach and Groin No. 1 on Treasure Island (Pinellas County, Florida), the groins for this project will have an effective length of roughly 1,400 feet to the north and 600 feet to the south of each structure. The length of each groin for the various berm widths are shown below:

Groin No. 1 length, ft	Groin No. 2 length, ft	Plan Berm width, ft
195	205	Nourishment only
220	230	25
245	255	50
270	280	75
295	305	100

Details of groin's design are developed in the following paragraphs.

d. Design wave.

(1) Table C-4 shows the relationships between H_o/gT^2, H_b/H_o', and d_b/H_b for a slope of m = 0.037. Figure 2-72, page 2-131 of the SPM (1984) defines the relationships between these variables. Hindcast deepwater waves from Gulf Station 40 (Hubertz and Brooks 1989) of 8.5 feet (5-year), 9.7 feet (10-year), and 10.4 feet (20-year) were used to compute a range of breaking waves, H_b. Wave periods ranging from 4 to 8 seconds were used. The still-water depth at the toe of the rubble groins was reviewed to determine if sufficient water depth existed at the toe of the structure to support the computed breaking wave. Table C-5 displays the data for depth at both the toe and crest of the groins.

(a) Review of the data in Tables C-4 and C-5 suggests that deepwater significant waves will break seaward of the structure. Therefore, the design wave will be depth limited. The nearshore slope seaward of the structure is m = 0.037 (1:27). It is assumed that the design wave for the stability of the quarry stone groin is the maximum wave that breaks directly on the structure. Since the elevation of the groins at the structure toe is +1.1 NGVD, the toe of the groins would not be subjected to breaking wave conditions. The portions of the groin towards the shore would be higher than the seaward portion. The landward groins sections would be the only

Table C-4
Breaking Wave Computations

	for H_o' = 8.5, ft*				for H_o' = 9.7, ft				for H_o' = 10.4, ft			
Wave Period (sec)	$\frac{H_o'}{gT^2}$	$\frac{H_b^*}{H_o'}$	H_b ft	** d_b ft	$\frac{H_o'}{gT^2}$	$\frac{H_b}{H_o'}$	H_b ft	d_b ft	$\frac{H_o'}{gT^2}$	$\frac{H_b}{H_o'}$	H_b ft	d_b ft
4.0	0.0165	1.48	12.6	16.1	0.0188	1.60	15.5	19.9	0.0202	1.64	17.1	21.8
6.0	0.0073	1.16	9.9	12.7	0.0084	1.18	11.4	14.7	0.0090	1.20	12.4	16.0
8.0	0.0041	1.08	9.2	11.8	0.0047	1.10	10.7	13.7	0.0050	1.11	11.5	14.8

* The values in this column are interpolated from Figure 2-73 of the SPM (1984).
** The values in this column are determined from the following relationship: depth of breaking is equal to 1.28 times the breaking wave (SPM 1984).

Table C-5
Total Water Depth at Structure Toe

Parameter	Return Period	Water Depth at Toe ft, NGVD	MHW Elevation ft, NGVD	Surge ft		Total Depth ft
Depth at toe	5	5.1	1.1*	3.7	=	8.8
(Depth at structure crest)		(1.2)	(1.1)*	(3.7)		(4.9)
Depth at toe	10	5.1	1.1*	4.9	=	10.0
(Depth at structure crest)		(1.2)	(1.1)*	(4.9)		(6.1)
Depth at toe	20	5.1	1.1*	6.2	=	11.3
(Depth at structure crest)		(1.2)	(1.1)*	(6.2)		(7.4)

* This value is already included in the surge water elevation. It is shown for information only.

sections that would have to resist the design breaking waves. Table C-5 shows the maximum water depth that could be expected at the crest elevation of the groins.

(b) Using Figure 7-4 of the SPM (1984), the maximum waves that break on the structure crest with d_s = 6.1 feet, nearshore slopes of 1:27, and wave periods from 4 to 1 seconds were determined as shown below.

gT^2 (sec)	$\frac{d_s}{gT^2}$	$\frac{H_b}{d_s}$	H_b (ft)
4	0.0118	0.95	5.80
6	0.0053	1.00	6.10
8	0.0030	1.05	6.41 (check)
10	0.0019	1.10	6.70 (check)
12	0.0013	1.12	6.80 (check)

(2) The check is to determine what effect underestimating the wave period will have on the breaker height. Based on a summary of deepwater wave hindcast data for all directions (Hubertz and Brooks 1989), waves occur 55.6 hours/year with periods greater than 6.5 seconds, or 6.3 percent of the time. Therefore, a breaking wave with wave periods between 4 to 6 seconds has been selected as the design wave. The design of the groins is based on a 6-foot broken wave acting on the shoreward portions of the groins.

e. Rock structure design.

(1) Rock structures. Two uniform-stone rock structures have been designed to hold the design beach-fill section in the southern end of the 4.2-mile-kilometer project area. Without the groins, the design fill would experience excessive losses of sand. Therefore, the

structures must be designed to be impervious to littoral material up to the design elevation of the beach fill, which is +5.0 feet (NGVD). The elevation of each structure varies along its length, as shown on Figure C-8. To make the groins impervious to sand, the groins will be constructed with a prestressed concrete sheet-pile core. Reinforced concrete or steel sheet piling may be substituted for the core of the groin, depending upon the results of the geotechnical subsurface investigations at the site. These investigations will be conducted during preparation of plans and specifications. The groins will be constructed with armor stone placed on both sides of the concrete sheet pile. The armor stone will protect the concrete sheet pile from wave attack. The armor stone will be placed on a foundation of bedding stone and filter cloth. Figure C-8 shows groin profile and cross-section details.

(2) Weight and slope of armor stone. The median weight and slope of the armor stone for the groin structures are designed in accordance with the SPM (1984). The median weight of the armor stone W_{50} of the groin structure is determined by Equation 7-116 of the SPM (1984): .

$$W = \frac{w_r H^3}{K_d (S_r - 1)^3 \cot \theta}$$

where

W_r = 165 pounds/cubic foot* (unit weight of armor stone)

H = 6.0 feet (design wave at structure head)

K_d = 1.6 (stability coefficient from Table 7-8, page 7-206 (SPM 1984), for breaking wave condition and two random layers of rough angular quarry stone at structure head)

S_r = w_r/w_w = 165/64 = 2.58 (specific gravity of armor unit)

w_w = 64.0 pounds/cubic foot (unit weight of water at the site)

cot Θ =2.0 (slope 1:2.0, angle of rock structure slope)

Substituting into the above equation yields an armor stone weight for the structure of 2,830 pounds** or 1.42 tons†.

* To convert pounds (mass) per cubic foot into kilograms per cubic meter, multiply by 16.01846.

** To convert pounds (mass) into kilograms, multiply by 0.4536.

† To convert tons (2,000 pounds, mass) into kilograms, multiply by 907.1847.

The range of armor stone weights for the cover layer of two quarry stones of the structure could vary from 0.75W to 1.25W (2,120 to 3,540 pounds) with about 50 percent of the individual stones weighing more than (2,830 pounds). A cross-sectional side slope of one vertical to two horizontal was selected.

(3) Armor layer crest thickness. The top width of the armor stone on both sides of the concrete sheet pile is a minimum thickness of two armor stones. The average thickness of armor stone layer r of the structure on each side of the concrete sheet pile is determined by Equation 7-121 (SPM 1984) as follows:

$$r = n\, k_\Delta \left(\frac{W}{w_r} \right)^{1/3}$$

where

n = 2 (layers of armor units)

k_Δ = 1.00 (layer coefficient from Table 7-13 (SPM 1984)

w_r = 165 pounds/cubic feet

W = 2,830 pounds

Substituting into the equation yields r = 5.16 feet. The rock structure would be constructed on a filter layer of cloth material. A layer of bedding stone would be placed on the filter cloth. The filter cloth and bedding stone act as a foundation for the armor stone. The bedding stone has a gradation of 1 to 50 pounds.

f. Foundation conditions.

(1) Groin No. 1. This groin is underlain by sand and silty sand, with no bedrock encountered to elevation -34.0 feet, MLW. Five feet of slightly cemented beach rock occur at elevations -2.1 to -7.1, but this layer has blow counts only slightly higher than the surrounding sand, with N values ranging from 9 to 17. This layer will not cause a problem driving the prestressed concrete sheet piles called for in the design.

(2) Groin No. 2. This groin is underlain by sand, with no bedrock encountered to elevation -35.6 feet, MLW.

(3) Both groins have riprap strewn over the nearshore surface resulting from existing groins and revetments in various states of disrepair. The core boring at Groins No. 2 encountered a piece of this riprap at elevation

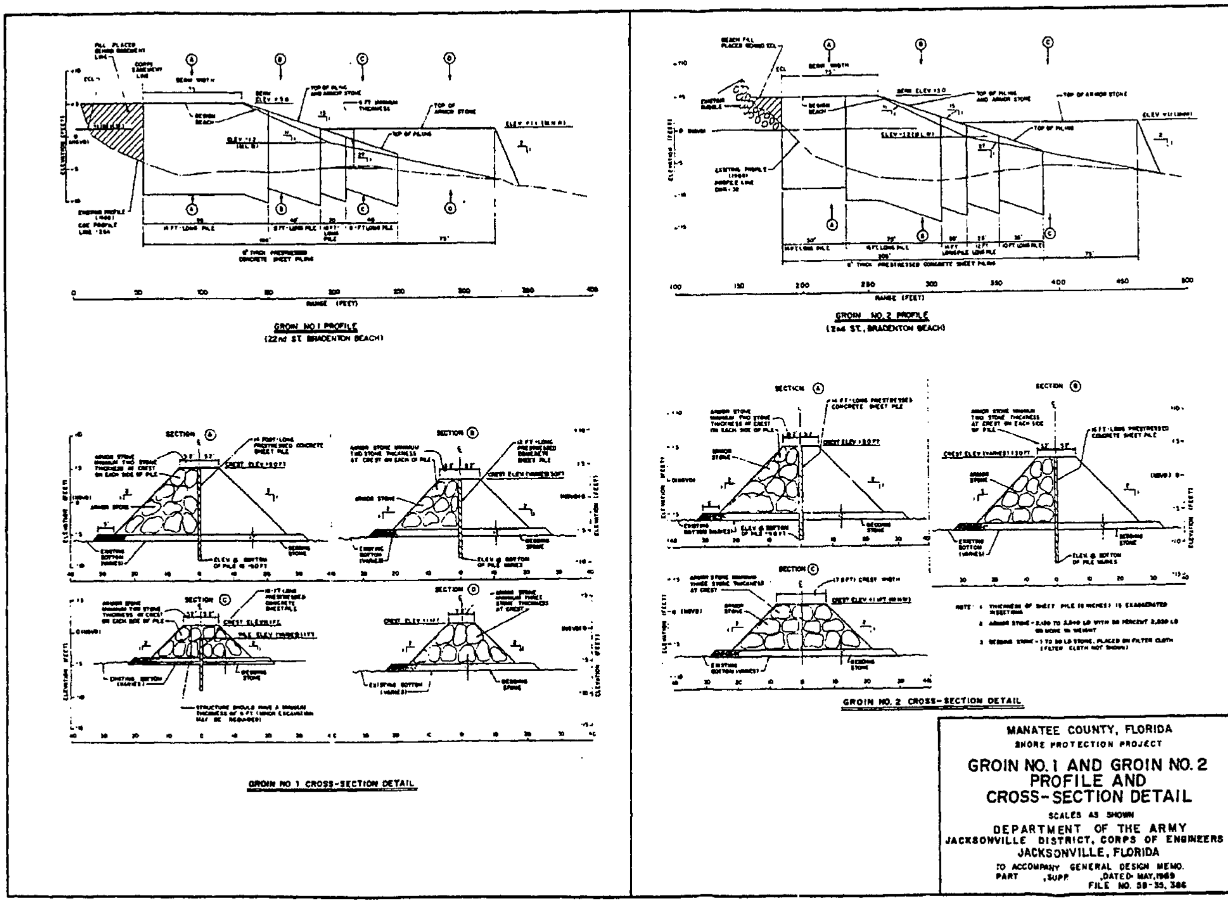


Figure C-8. Groin No. 1 and Groin No. 2 profile and cross-section detail, Manatee County, FL (USAED, Jacksonville 1990)

PLATE 12

-10.0 MLW, and it is safe to assume that scattered riprap occurs throughout the sand column.

g. Predicting future maintenance. Using Table 7-9, page 7-211 of the SPM (1984), the damage that can be expected if the design wave is exceeded can be determined. The future maintenance of the groins can then be estimated. The groins have been designed to withstand a 10-year storm significant wave event with less than 5-percent damage. A maintenance interval of 10 years has been selected. There is a 40-percent probability that a 20-year surge event will occur in a 10-year period. This surge would result in a design depth at the crest of the structure crest of 7.4 feet. Using a wave period of 8 seconds and Figure 7-4 of the SPM, d_s/gt^2 = 0.0036, and H_b/d_s = 1.05. Therefore, H_b = 7.8 feet, and H/H_D = 7.8/6.0 = 1.30 percent, which from Table 7-9 of the SPM indicates between 10- to 22-percent damage to the cover layer. There is an 18-percent probability that a 50-year surge event will occur in a 10-year interval. Therefore, the damage caused by this event was not considered in the maintenance of the groins. A factor of 20-percent damage to the armor layer every 10 years was used to determine the cost of groin maintenance.

Appendix D GENESIS Numerical Shoreline Change Model

D-1. GENESIS Numerical Shoreline Change Model

A numerical modeling system called GENESIS has been designed to simulate long-term shoreline change at coastal engineering projects such as groins, offshore breakwaters, seawalls, and beach fills (Hanson and Kraus 1989*). The name GENESIS is an acronym that stands for GENEralized Model for SImulating Shoreline Change.

D-2. Application

a. Input data. Input data to GENESIS include the initial shoreline location and wave conditions at either an offshore (deepwater) location or in a given water depth seaward of the expected breaking depth at closely spaced stations along the reach of shoreline being investigated. If offshore wave data are used, the program simply assumes straight and parallel bottom contours to transform waves from deep water to the breaking point. If nearshore bottom contours are complex and refraction transformations cannot be approximated by straight and parallel contours, then a wave transformation program, RCPWAVE (Ebersole, Cialone, and Prater 1986), is used to transform waves from the deepwater location to a nearshore water depth seaward of the wave's breaking depth. GENESIS then transforms the waves shoreward from this nearshore depth to breaking using straight and parallel bottom contours. If RCPWAVE is used, approximately 100 offshore wave conditions within various period and direction classes are selected from the actual wave climate (usually determined from Wave Information Study (WIS) hindcasts) so that they approximate the actual range of directions and periods at the site under investigation. These conditions are run using RCPWAVE to compute a table of wave height factors (the ratio of breaker height to deepwater wave height) for the given climate of wave directions and periods at the nearshore location. In running GENESIS, the offshore wave period, direction, and height are determined from the wave climate for the area (WIS hindcast), and the local direction and height are determined for each nearshore RCPWAVE station. The local direction is found from the table generated by RCPWAVE for the given offshore direction and period, and the local height is calculated from the tabulated RCPWAVE wave height factor.

b. Calculations. Longshore transport is calculated at fixed locations along the shoreline (submultiples of the nearshore RCPWAVE stations) using a modified form of Equation 2-7 (see the main text), which includes a term for the transport resulting from any longshore gradient of the breaking wave height (diffraction term). This equation is given by,

$$Q = \frac{\left(H^2 C_g\right)_b}{8\left(\frac{\rho_s}{\rho} - 1\right)\hat{a}\ (1.416)} \left[\frac{k_1}{2}\sin 2\theta_b - \frac{k_2}{1.416}\frac{\cos\theta_b}{\tan\beta}\frac{H}{x}\right]_b \tag{D-3}$$

in which

H = wave height

C_g = wave group speed given by linear wave theory

$\hat{a}$ = 1 - porosity of the in situ sand on the beach (taken to be 0.6)

ρ_s = density of sand

ρ = density of water

Θ_b = angle breaking wave makes with the local shoreline

k_1 and k_2 = empirical coefficients

$\tan \beta$ = average bottom slope across the surf zone out to the depth of active longshore sand transport

Two coefficients enter into Equation D-1; k_1 is the usual coefficient of proportionality relating transport rate with longshore energy flux, and k_2 is a coefficient for the longshore gradient of the breaking wave height term. Both coefficients may be adjusted to calibrate the model against observed shoreline changes at a site.

c. Groins. GENESIS can consider the effects of groins, nearshore breakwaters, seawalls, and beach fills on the shoreline. These structures impose local boundary conditions within the reach of shoreline under investigation. Groins are subdivided into either nondiffracting or diffracting. Generally, nondiffracting groins are relatively short whereas diffracting groins are long. GENESIS allows the user to specify a permeability for each groin which is one of two factors that determines how much sand bypasses the groin. Groin permeability can also be adjusted and used as a calibration factor to fit the model with observed

* See References at the end of the main text.

prototype shoreline changes. The other factor governing the amount of sand bypassing a groin is the location of the breaker line relative to the seaward end of the groin. Sand is assumed to be in longshore transport along the beach out to a depth of about 1.6 times the breaking depth of the transformed significant wave height (Hallermeier 1983). Consequently, if the depth of longshore sediment transport extends beyond the end of the groin, some sand will pass around the end of the groin. The cross-shore distribution of longshore transport is assumed to be uniform so that the amount of sand passing around the groin's end is the ratio of the distance beyond the end of the groin out to the point where longshore transport occurs to the distance from the shoreline out to this point.

d. Breakwaters. GENESIS treats detached breakwaters as diffracting structures with wave energy propagating around each end. Wave heights decrease as one moves farther behind the breakwater; however, waves generally propagate around each end. Each end of the breakwater defines an "energy window," and longshore transport is computed at each shoreline point using the wave energy propagating through the two windows, one defined by each end of the breakwater. In addition, wave transmission over and through the breakwater can be included. In general, tombolo formation is precluded if sufficient wave energy is transmitted into the sheltered area behind the breakwater. Allowing energy transmission over and/or through a breakwater can be an important design consideration. The transmission coefficient provides an additional factor that can be used to calibrate the GENESIS model. The guidelines for salient and tombolo formation given by Hanson and Kraus (1990) were determined by numerous runs of GENESIS with various wave climates, breakwater lengths, wave transmission coefficients, and offshore distances.

e. Seawalls and bulkheads. GENESIS is also capable of predicting shoreline changes in the vicinity of seawalls and bulkheads. Erosion of a stretch of beach is halted when the shoreline retreats back to the seawall or bulkhead. The longshore transport in front of the seawall then becomes constant as determined by the transport at the first "non-hardened" point updrift of the seawall.

f. Boundary conditions. Various boundary conditions are allowed at the edge of the GENESIS model. The two most important impose constraints either on the shoreline's location at the model's boundary, or on the quantity of sediment entering and/or leaving the model. The first is termed a "fixed end" boundary where the shoreline is held fixed at a point. This is equivalent to having the transport into the end cell balance the transport out of the end cell so that no net accretion or erosion occurs. The second is termed a "gated" boundary where the shoreline orientation is fixed so that the rate of sediment entering or leaving the model is constant.

g. Evaluation of shore stabilization schemes. GENESIS provides a useful tool in evaluating proposed shore stabilization schemes. Numerous alternatives can be studied, and the shoreline changes caused by them estimated. For example, shoreline effects from combinations of groin spacing and length can be investigated, or breakwater lengths, spacing, and distances from shore can be studied. A typical design procedure would involve initially selecting a promising alternative using empirical guidance, followed by a detailed evaluation of that alternative using GENESIS and RCPWAVE. The alternative could then be modified to more closely achieve the desired shoreline response. The modification-evaluation procedure is continued until the desired shoreline is predicted by GENESIS. Some example areas to which GENESIS has been applied include: Homer Spit, AK; Sandy Hook to Manasquan, NJ; Bolsa Chica, CA; Canaveral Inlet, FL; Brazos River Diversion, TX; Folly Beach, SC; and Lakeview Park, Lorain, OH.

Appendix E
Dimensional Analysis for Nearshore Breakwaters and Example Application

E-1. Dimensional Analysis for Detached Breakwaters

Dimensional analysis can provide some insight into the design of single and multiple detached breakwater systems. A simplified picture of a single detached breakwater is given in Figure E-1 along with important variables that describe a typical design problem.

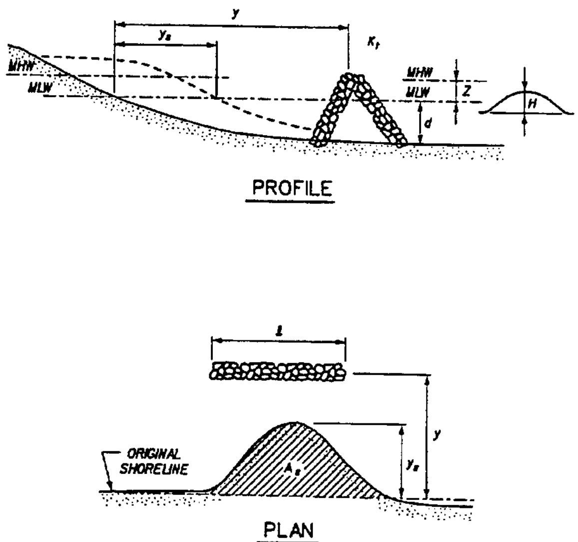


Figure E-1. Shoreline evolution behind a single detached breakwater and definition of terminology (MHW) = high water; MLW = mean low water)

a. Variables for single breakwater. For a single breakwater, the variables with their dimensions (in square brackets) are:

ℓ = breakwater length, [L]

y = distance from the average shoreline, [L]

y_s = distance to end of the salient from the average shoreline, [L]

H_b = breaking height of a characteristic breakwater design wave, [L]

d = water depth at the breakwater, [L]

d_b = breaking depth of the characteristic design wave, [L]

T = wave period, [T]

g = acceleration of gravity, $[L]/[T]^2$

K_t = wave transmission coefficient, [dimensionless]

z = tidal range, [L]

A_s = beach planform area within salient, $[L]^2$

x = distance along shore, [L]

t = time, [T]

One set of dimensionless variables that can be obtained from a dimensional analysis is given by,

$\pi 1 = \dfrac{\ell}{gT^2}$ = dimensionless breakwater length

$\pi 2 = \dfrac{y}{\ell}$ = dimensionless distance offshore of breakwater

$\pi 3 = \dfrac{y_s}{y}$ = dimensionless salient length

$\pi 4 = \dfrac{H_b}{d}$ = wave-height-to-water-depth ratio

$\pi 5 = \dfrac{d}{d_b}$ = dimensionless water depth at the breakwater

$\pi 6 = \dfrac{H_b}{gT^2}$ = breaking wave steepness

$\pi 7 = \dfrac{z}{d}$ = relative tidal range

$\pi 8 = \dfrac{A_s}{y\ell}$ = dimensionless salient area

$\pi 9 = \dfrac{H}{\ell}$ = dimensionless distance measured alongshore

$\pi 10 = \dfrac{t}{T}$ = dimensionless time (number of waves)

$\pi 11 = K_t$ = dimensionless wave transmission coefficient.

b. Variables for breakwaters separated by gaps. The dimensionless variables that have been given are not unique. Other combinations of terms are possible. Figure E-2 depicts the situation where several breakwaters are separated by gaps. Three additional variables might be included. They are,

b = gap width [L],

y_g = the shoreline recession from the average shoreline behind the breakwater gap, [L]

A_g = the area in the shoreline recession behind the average shoreline behind the breakwater gap, $[L]^2$

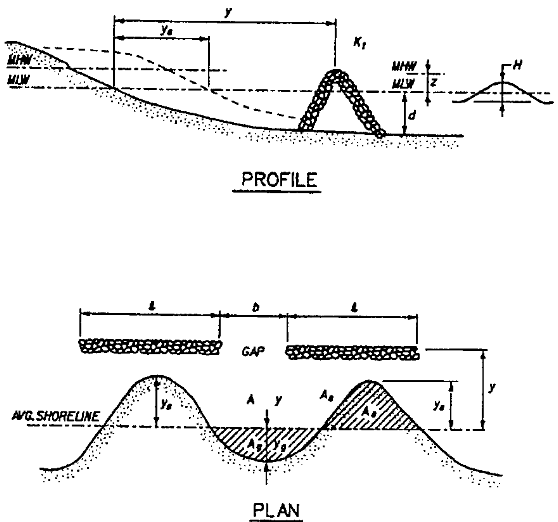


Figure E-2. Shoreline evolution behind a multiple nearshore breakwater system and definition of terminology

These additional terms lead to three additional dimensionless variables,

$\pi 12 = \frac{y_g}{y}$ = dimensionless gap indentation length

$\pi 13 = \frac{A_g}{by}$ = dimensionless area of shoreline indentation behind breakwater gap

$\pi 14 = \frac{b}{\ell + b}$ = "exposure ratio" (the fraction of the shoreline exposed to direct action of incident waves)

Some typical "exposure ratios" for existing breakwater systems are given in Table 4-1 (see the main text). Alternatively, a "sheltering ratio" could be defined as,

$\pi 14' = \frac{\ell}{\ell + b}$ = "sheltering ratio"

The "exposure ratio" and "sheltering ratio" are not independent of each other since their sum must equal 1.

c. Dimensionless parameters for single and multiple breakwaters.

(1) If longshore transport is also included in the analysis, an additional dimensionless variable can be defined. A simple dimensionless variable might be,

$\pi 15 = \frac{Q_n}{H_b^3/T}$ = dimensionless transport rate

where

Q_n =longshore transport rate behind the nearshore breakwater system, $[L]^3/[T]$

(2) An alternative and perhaps more physically meaningful dimensionless variable can be obtained by making use of the Coastal Engineering Research Center (CERC) longshore transport equation (Equation 2-8) to normalize Q. For example,

$\pi 15' = \frac{Q_n (\rho_s - \rho)\, \hat{a}}{0.0055\, \rho\, H_b^{5/2} g^{1/2} \sin 2\Theta_b}$ = ratio of transport rate behind breakwater system to transport rate on an unobstructed beach

(3) By introducing the CERC formula, four additional variables have been introduced, two of which are already dimensionless. The variables are:

ρ_s = mass density of the sediment, $[M]/[L]^3$

ρ = mass density of water, $[M]/[L]^3$

$\hat{a}$ = solids fraction of the in situ sediment deposit (dimensionless)

Θ_b = angle the breaking waves make with the shoreline in the absence of the breakwater system (dimensionless)

(4) Only one additional dimensionless variable must be added since an additional dimension, mass, has been added. For example,

$$\pi 16 = \frac{\rho_s}{\rho} = \text{ratio of the sediment's mass density to the water's mass density}$$

Since there is little variation in the unit weight of the sediments, $\pi 16$ is approximately constant.

(5) The dimensionless breakwater length, $\pi 1 = l/gT^2$, can be taken as a scaling factor that can be used to transpose observations of breakwater performance from one location to another. For example, the average wave period along the Gulf of Mexico coastline of the United States is about 5.5 seconds. A breakwater 200 feet* long would have a dimensionless length of $l/gT^2 = 0.206$. If this were to be compared with a breakwater project on the Atlantic coast of the United States where the wave period is about 7.0 seconds, the corresponding breakwater length would be about $= 0.206\ gT^2 = 325$ feet. If the distance from shore in the gulf were 100 feet, the corresponding distance from shore in the Atlantic would be 163 feet. The average wave period for Pacific coast beaches of the United States is 12.5 seconds. Thus the 200-foot-long breakwater, 100 feet from shore in the gulf would scale up to a 1,035-foot-long breakwater 518 feet from shore on the Pacific coast.

(6) $\pi 2$ is the dimensionless distance of the detached breakwater from shoreline. The inverse of $\pi 2$ appears to be the single factor most important in determining whether a tombolo or a salient forms behind the breakwater.

(7) $\pi 3$ is a dimensionless salient length that takes on values between 0 and 1, $\pi 3 = 1.0$ for a tombolo.

(8) $\pi 4$ is a dimensionless breaking wave height that also determines if the breakwater is inside or outside the surf zone. If $\pi 4$ is less than about 0.78, the breaker line will be landward of the breakwater. For $\pi 4$ greater than 0.78, the breaker line will be seaward of the breakwater; i.e., waves will break before they reach the breakwater and the breakwater will be within the surf zone.

(9) Similarly, $\pi 5$ is the dimensionless water depth at the breakwater. If $\pi 5 < 1.0$, the breakwater lies within the surf zone, and waves break seaward of the breakwater. If $\pi 5 > 1.0$, the breakwater is seaward of the surf zone, and waves break landward of the breakwater. The product of $\pi 4$ and $\pi 5$ is the breaking wave height to breaking depthratio and is usually about 0.78, although there is some dependence of this ratio on beach slope.

(10) $\pi 6$ is the breaking wave steepness and is a measure of the wave environment at the site.

(11) $\pi 7$ is the dimensionless tidal range; it is a measure of how much the water depth changes at the breakwater over a tidal cycle.

(12) $\pi 8$ is a measure of how much sand accumulates in the salient behind a breakwater. It is the fraction of the area behind the breakwater that lies within the salient. It is thus generally less than 1.0. Smaller values of $\pi 8$ indicate smaller volumes of accumulation within the salient; they do not necessarily imply a less effective breakwater system, however, since the shoreline might be stabilized without developing salients. Values of $\pi 8$ approaching 1.0 indicate tombolo formation. $\pi 9$ and $\pi 10$ are dimensionless independent variables representing the distance alongshore and the time (number of waves), respectively.

(13) $\pi 11$ is the breakwater's wave transmission coefficient. It is important in determining whether or not a tombolo forms. Breakwaters that allow significant amounts of wave energy to be transmitted over or through them are less likely to have tombolos form.

d. Dimensionless parameters for multiple breakwater systems. The preceding dimensionless parameters can be defined for both single breakwaters and for breakwater systems. The following dimensionless parameters are defined only for multiple breakwater systems.

(1) $\pi 12$ is the dimensionless shoreline indentation in back of the gap between two adjacent breakwaters.

(2) $\pi 13$ is the dimensionless area of the shoreline indentation behind the breakwater gap. If the average postconstruction shoreline is defined as the shoreline that balances erosion behind the gaps against accretion behind the breakwaters, the value of Ag will be approximately equal to As Thus, there is a relationship between $\pi 8$, $\pi 13$, and $\pi 14$ given by $\pi 13 = \pi 8\ (1/\pi 14 - 1)$, where $\pi 14$ is the "exposure ratio" defined in the following paragraph.

(3) $\pi 14$ is the dimensionless "exposure ratio." It represents the fraction of the shoreline exposed to waves propagating through the breakwater gaps. Values of $\pi 14$ greater than about 0.5 indicate relatively large gaps with gaps that are longer than the breakwaters. Values of $\pi 14$ less than 0.5 are more typical of prototype installations as

* To convert feet into meters, multiply by 0.3048.

indicated in Table 4-1 (see the main text), which gives "exposure ratios" for several prototype breakwater installations. Alternatively, π14' is a "sheltering ratio" that represents the fraction of the shoreline sheltered from incoming waves by the breakwaters. π14 and π14' are related by the expression (π14 + π14') = 1.0 and are thus not independent of each other.

(4) π15 and π15' are dimensionless potential sediment transport rates. Their effect on the performance of nearshore breakwaters has not been documented, but since nearshore breakwaters interrupt longshore transport, they measure how rapidly sediment is transported through a system of nearshore breakwaters and how rapidly a system of breakwaters traps sand. This is important if beach fill is not a part of a nearshore breakwater project or if a given amount of sediment transport through a breakwater system is to be maintained. π16 is simply the ratio of the sediment's density to the water's density. While this is relatively constant in the prototype at about 2.65, moveable bed models may use materials other than quartz sand. If this is the case, the fall velocity of the sediment becomes important in interpreting the results of the model tests. In fact, the mean sediment diameter is also important, and the following dimensionless parameters arise.

$$\pi 17 = \frac{V\,T}{H_b} = \text{dimensionless sediment fall velocity}$$

$$\pi 18 = \frac{D_{50}}{d} = \text{dimensionless sediment diameter}$$

in which V = the fall velocity of the sediment (the terminal velocity at which an "average" sediment grain will fall through a water column) and D_{50} = the mean diameter of a sediment grain.

E-2. Example Application

a. Problem. The empirical relationships and design procedures can be applied to the hypothetical design problem for Ocean City, NJ, started in Appendix C. The problem is to stabilize an 18-block-long reach of the beaches between 17th Street in the north and 36th Street in the south. A location map is given in Figures C-2a, b, and c. The objective is to provide a minimum berm width of 100 feet measured seaward from the existing bulkhead line by providing beach nourishment. Nearshore breakwaters are to be evaluated as a means of retaining the beach nourishment within the project area. Some longshore transport is to be maintained after the breakwaters have been built to minimize any potential erosion downdrift and updrift of the project. Typical beach profiles are given in Figures C-3 and C-4.

b. Japanese Ministry of Construction procedure.

(1) The Japanese Ministry of Construction (JMC) procedure will be applied first. From Figure 2-14 (see the main text), the wave height exceeded at least once a year is about 2.5 meters (8.2 feet). From the analysis in Chapter 3, the weighted average wave height is 2.1 feet with a period of 6.5 seconds. Both of these wave heights are given in a water depth of 10 meters (32.8 feet). For the purposes of the present problem, the H_5 wave height will be selected as the average of the 1-year wave height and the annual average wave height; thus, $H_5 = (2.1 + 8.2) = 5.15$ feet and $T_5 = 6.5$ seconds. Lo_5, the deepwater wavelength associated with the H_5 wave, is $5.12\ (6.5)^2 = 216$ feet, and from a shoaling analysis, the deepwater height of the H_5 wave is $H_{o5} = 5.63$ feet. By comparison with the above descriptions of the shoreline types, Ocean City's shoreline most closely approximates the conditions describing a Type B shoreline since the beach slope at Ocean City is about 1:40 and the wave height exceeds 0.5 meters (1.64 feet).

(2) Entering Figure 4-7 with the ratio $Ho_5/L_{o5} = 0.026$, the ratio $d/H_{o5} = 1.6$ is found and $d = 1.6\ (5.63) = 8.95$ feet. An initial value of the salient extension of $y_s = 150$ is selected. The present water depth at the end of the salient is $d = y_s \tan\beta = 150/40 = 3.75$ feet. The estimated water depth at the breakwater is $3.75 < d' < 8.95$ or, taking the average $d' = (3.75 + 8.95)/2 = 6.35$ feet. Then $d'/d_b = 6.35/8.95 = 0.75$. Entering Figure 4-8 with $d'/d_b = 0.71$ gives salient area ratio (SAR) = 0.75. The distance offshore of the breakwater is $y = d'/\tan\beta = 6.35\ (40) = 254$ feet. The distance of the salient extension, y_s = SAR $y = 0.71\ (254) = 190$ feet which is greater than the originally selected value of 150 feet. Consequently, another iteration should be made based on a new guess of y_s. For the current example, subsequent iterations using both larger and smaller initial values of y_s did not converge. Instead, the value of d' was not determined from an average of the existing water depth at the projected end of the salient and the breaking depth, but rather a value closer to the lower end of the range was selected. Thus, instead of selecting $d' = 6.35$ feet, a value of 5.5 feet was selected. Thus, $d'/d_b = 5.5/8.95 = 0.61$ which yields a value of SAR = 0.7. The distance offshore of the breakwater is $5.5/\tan\beta = 220$ feet, and the salient extension is $y_s = y$ SAR $= 220(0.7) = 154$ feet, which is approximately equal to the initially selected value of $y_s = 150$ feet.

(3) The breakwater length is determined from Equations 4-6 and 4-8 for a Type B shoreline. The wavelength of the design wave at the proposed breakwater

is given by $L_5 = T\sqrt{(gd)} = 6.5\sqrt{(32.17)(5.5)} = 86.5$ feet. From Equation 4-6, $1.8\ L_5 < \ell < 3.0\ L_5$, or $156 < \ell < 259$. From Equation 4-8, $0.8\ y < \ell < 2.5\ y$, or $176 < \ell < 550$. The average of the maximum minimum, 176, and the minimum maximum, 259, yields a breakwater length of 217.5 feet, say 220 feet.

(4) The gap between breakwaters is determined from Equations 4-10 and 4-11. The gap length is given by $0.7\ y < b < 1.8\ y$, or $154 < b < 396$. Also, $0.5\ L_5 < b < 1.0\ L_5$, or $43.2 < b < 86.5$. These two ranges are mutually exclusive; however, an estimate of the gap width is again the average of the minimum maximum, 86.5, and the maximum minimum, 154. Thus, b = (86.5 + 154)/2 = 120 feet.

c. Other possible breakwater systems.

(1) Two other possible breakwater systems were investigated and are summarized in Table E-1.

Table E-1
Summary of Nearshore Breakwater Systems Evaluated for Ocean City, NJ

y, ft	y, ft	d, ft	L_5, ft	SAR	ℓ, ft	b, ft
150	220	5.5	86	0.70	220	120
100	180	4.5	78	0.55	190	100
50	132	3.3	67	0.40	160	80

(2) Several other empirical relationships presented in Chapter 4 can also be used to determine breakwater length, distance from shore, and gap width. Selecting the design wave height as the mean annual wave height, $H = 3.0$ feet and $T = 6.5$ seconds (see Chapter 3). The water depth at breaking is approximately 3.0/0.78 = 3.84 feet. Since the beach slope is approximately 1:40 and if the breakwater is located at the breaking depth of the mean annual wave, the breakwater will be located $y = 3.84(40) = 153$ feet - say 150 feet - from shore.

(3) From Table 4-3 using a conservative estimate of $\ell/y < 0.5$ to preclude tombolo formation, the breakwater length is $\ell < 0.5(153) = 77$ feet, say 80 feet. The gap width can be estimated from Suh and Dalrymple's (1987) relationship in Table 4-4. Rearranging the equation gives,

$$b \geq 1/2\ \ell_2/y \qquad \text{(E-1)}$$

(4) Then, $b \geq 0.5\ (80)^2/150 = 21$ feet. Thus the gap must be more than 21 feet wide to prevent tombolo formation. Use $b = 40$ feet.

(5) The salient extension can be estimated from Suh and Dalrymple's relationship given in Equation 4-4. Equation 4-4 gives $y_s = 0.89y = 0.89(150) = 134$ feet. This represents a rather pronounced salient. The results are summarized below.

y = 150 feet

ℓ = 80 feet

b = 40 feet

y = 134 feet

All of the preceding designs must be considered preliminary and would have to be studied further and refined using either physical or numerical model studies.

E-3. Dimensional Analysis for a Submerged Sill

a. Variables. A dimensional analysis of the perched beach yields the following variables given with their dimensions (Figure E-3).

d_s = water depth at the sill structure measured on the landward side, [L]

d_{ss} = water depth at the sill structure measured on the seaward side, [L]

h_s = height of the sill crest above the bottom measured on the landward side,[L]

h_{ss} = height of the sill crest above the bottom measured on the seaward side [L]

y_s = distance from the sill to the mean low water (MLW) shoreline, [L]

f = water depth over the crest of the sill measured from the MLW line, [L]

H = wave height measured at the seaward side of the sill, [L]

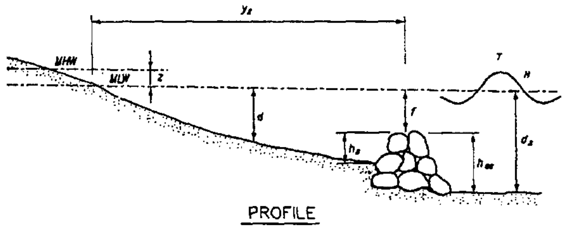


Figure E-3. A submerged sill system and definition of terminology

T = wave period, [T]

g = acceleration of gravity, [L]/[T]^2

z = mean tidal range, [L]

V = fall velocity of the median sand grains, [L]/[T]

ρ_s = sediment density, [M]/[L]^3

ρ = fluid density, [M]/[L]^3

y = horizontal distance measured landward from the sill crest (an independent variable), [L]

d = local water depth measured from the MLW line - a function of y, [L]

K_t = wave transmission coefficient (dimensionless)

b. Dimensionless π terms.

(1) The 16 variables can be combined into 13 dimensionless π terms. There are two equations that result from the sill structure's geometry that relate the variables; hence, the problem can be reduced to 11 dimensionless terms. These equations are $h_s + f = d_s$ and $h_{ss} + f = d_{ss}$. The original 13 π terms are:

$\pi 1 = K_t$ = wave transmission coefficient

$\pi 2 = \dfrac{\rho_s}{\rho}$ = relative sediment density

$\pi 3 = \dfrac{H}{gT^2}$ = wave steepness at the sill

$\pi 4 = \dfrac{d_{ss}}{gT^2}$ = relative water depth on the seaward side of the sill

$\pi 5 = \dfrac{f}{d_{ss}}$ = dimensionless depth over the sill crest

$\pi 6 = \dfrac{d_s}{y_s}$ = average slope across the profile on the landward side of the sill

$\pi 7 = \dfrac{d_s}{d_{ss}}$ = discontinuity in the beach profile at the sill

$\pi 8 = \dfrac{y}{y_s}$ = dimensionless distance measured landward from the sill (dimensionless independent variable)

$\pi 9 = \dfrac{d}{d_s}$ = dimensionless depth - a function of π7

$\pi 10 = \dfrac{VT}{H}$ = dimensionless fall velocity of median sand grain

$\pi 11 = \dfrac{z}{d_{ss}}$ = relative tidal range

$\pi 12 = \dfrac{h_s}{d_s}$ = dimensionless sill height measured on the landward side of the sill

$\pi 13 = \dfrac{h_{ss}}{d_{ss}}$ = dimensionless sill height measured on the seaward side of the sill

(2) The two equations allow π12 and π13 to be expressed in terms of the other dimensionless π terms. Thus, π12 = π7 - π5, and π13 = 1 - π5.

(3) As more experience with perched beaches accumulates, the preceding dimensionless terms can be used to relate the behavior of various installations to each other. Unfortunately, there is currently little experience on which to base a design.